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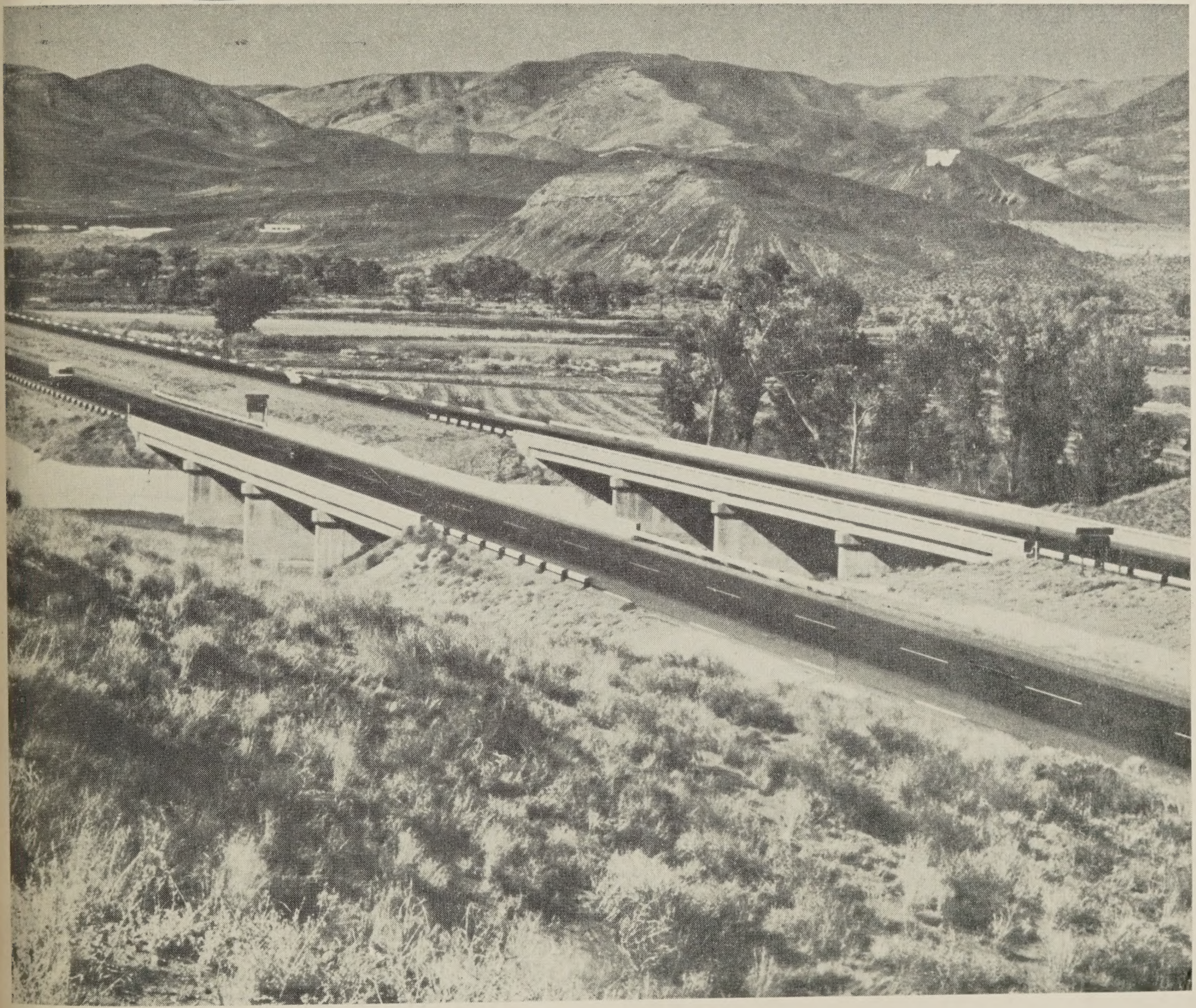

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
BUREAU OF PUBLIC ROADS

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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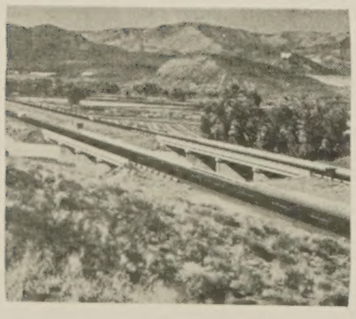

COVER

Twin bridge structures on a completed section of Interstate Highway 80 east of Reno, Nev.

U.S. DEPARTMENT OF TRANSPORTATION
JOHN A. VOLPE, Secretary
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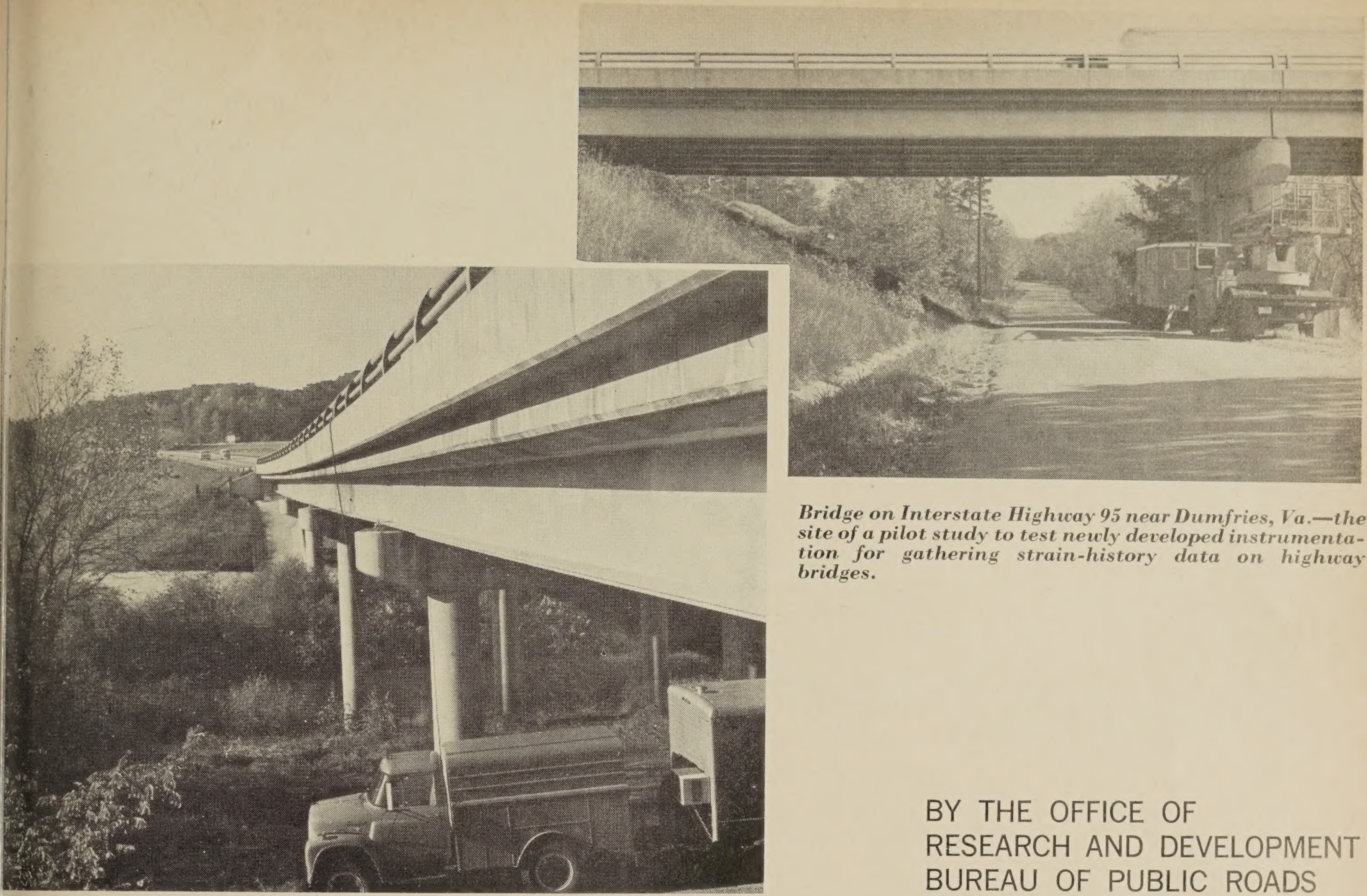

Bridge on Interstate Highway 95 near Dumfries, Va.—the site of a pilot study to test newly developed instrumentation for gathering strain-history data on highway bridges.

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Acquisition of Loading History Data on Highway Bridges

ported by [1] CHARLES F. GALAMBOS
d WILLIAM L. ARMSTRONG, Struc-
ral Research Engineers, Structures
d Applied Mechanics Division

A data acquisition system to monitor and digitize strains produced in highway bridges by ordinary truck traffic is described in this article. The system and an associated computer program were used in a pilot study, on an Interstate Highway bridge near Dumfries, Va. Strains were monitored during several sampling periods, and trucks were weighed and classified at a nearby weighing station. The performance of the system was satisfactory. Generally low strains, usually less than half of the calculated live-load design values, were measured and elements of the Dumfries bridge seem to be in no danger of fatigue failure.

Introduction

MUCH of the conservative approach to the design of highway bridges is due to the predictability of the precise nature of the dings to which these structures will be bjected in their lifetime, and also to the k of knowledge about what the behavior the materials will be under the loadings.

Highway bridges are subjected to a variety forces, ranging from the constant dead loads the structures themselves, through slowly anging forces caused by material creep and mperature differentials, to an almost infinite variety of live loadings caused by moving vehicles. It would be impossible to base bridge designs on any precise knowledge of future loads, and it follows naturally that present design methods are approximations. Most bridges are designed to carry a static load produced by a design truck, with certain empirical allowances for increased stresses owing to dynamic loads.

Design methods provide for the damaging effects of repetitive loads only in a rather crude fashion—a fact that is becoming of increasing concern to bridge engineers. There is an obvious need to determine the loading history of highway bridges so that the stresses produced by traffic can be predicted and a more rational, realistic estimate of future stresses made.

Attempts at monitoring the stresses caused by traffic traversing a bridge have been cumbersome and time consuming because of laborious data reduction processes. The statis-

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Presented at the 48th annual meeting of the Highway search Board, Washington, D.C., January 1969.

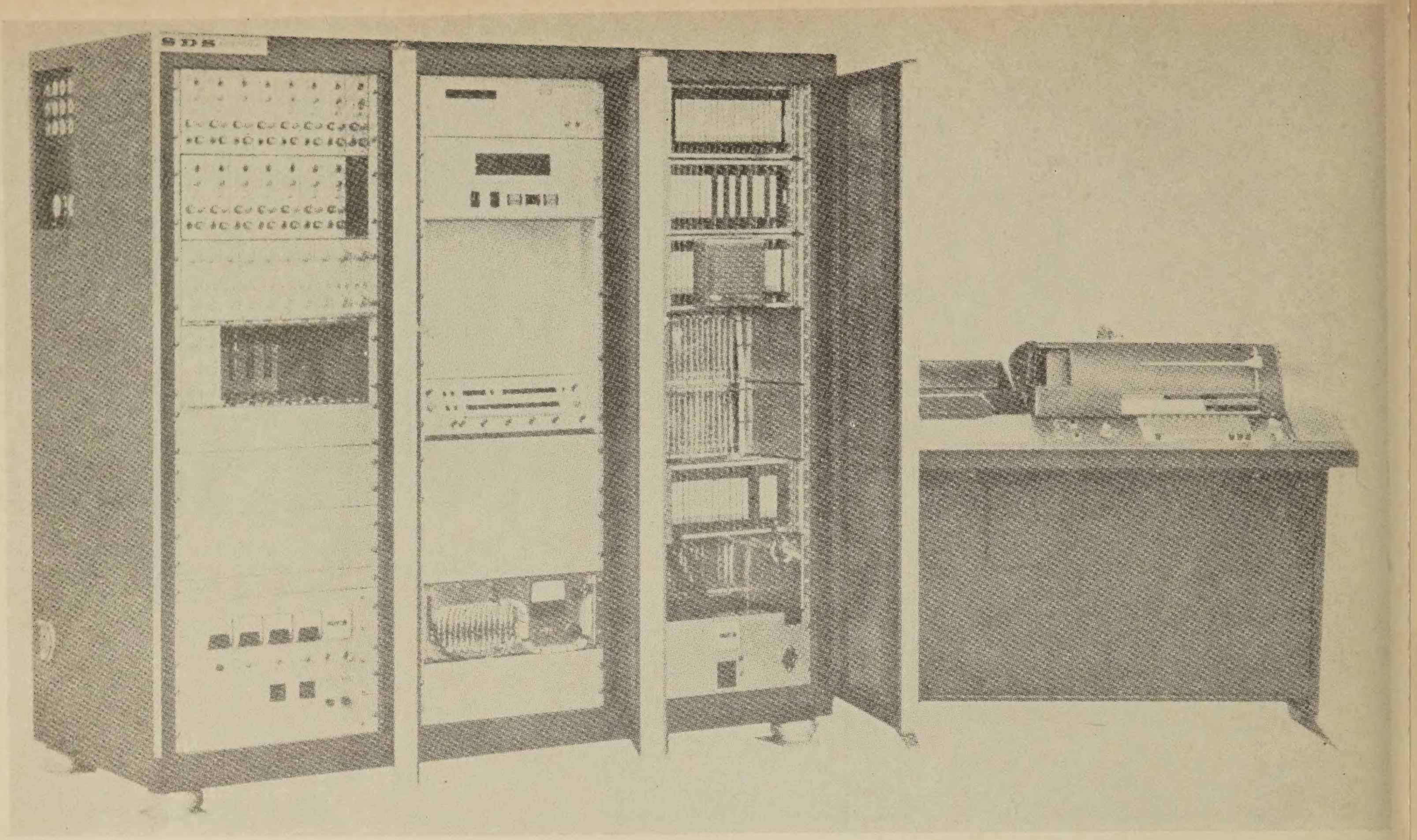


Figure 1.—Strain-history-data acquisition system.

tical nature of the problem requires that a large mass of data be collected. Enough information must be gathered on various types of bridges in representative parts of the country so that the loading histories described will be representative of what really takes place.

The Public Roads staff, having been active in the field testing of highway bridges since 1953, is well aware of the problems involved in the data reduction process and has shared in the concern of the need for better instrumentation. Accordingly, the Structures and Applied Mechanics Division of the Office of Research and Development assembled a set of specifications and employed a contractor to develop an instrumentation system to *monitor, digitize, and record physical phenomena of highway bridges in service.* Work on the project began in the summer of 1965, and the system was completed in May of 1966. Since then it has been subjected to an extensive acceptance test and used in a pilot study on a bridge near Dumfries, Va.

Description of Instrumentation

Detailed components of system

The data acquisition system is an assemblage of instrumentation consisting largely of signal conditioning modules, amplifiers, an analog-to-digital converter, a digital processing unit (computer), and an input-output device (teletype with paper punch). Power supplies, cooling fans, and a voltage regulator complete the assembly, and except for the regulator and the teletype, all units are housed in shock-mounted cabinets. The entire assembly is housed in an office trailer—not part of the development contract—that can be easily transported. Heating and cooling units automatically control the temperature and humidity in the trailer. A view of the instrumentation is shown in figure 1, and a system block diagram is presented in figure 2.

Although electric power to the system can be supplied from portable AC generators, power from the more reliable commercial power lines is preferable. A single-phase, three-wire power source of approximately 40 amperes is required. Except for the air cooling unit in the trailer, which requires 220 volts, all equipment operates on 110-volt power.

Signal conditioning modules

The transducer input circuits from the bridge structure being measured are completed in the signal conditioning modules. Inputs from one-, two-, or four-arm Wheatstone bridges are acceptable. Also, a variety of interchangeable completion cards are available for use with transducers other than strain gages, such as thermocouples, thermistors, and potentiometers. Adjustments have been provided for balancing the signal conditioning circuits and for calibration.

Amplifiers

The DC amplifier multiplexers are interchangeable units, specially designed for use in multichannel, low-level, data acquisition systems. Each unit consists of an input amplifier, modulator, demodulator, isolator, wide-band output amplifier, extra output amplifier with an active filter, and multiplexer circuit. The input range is ±10 millivolts through ±10 volts for a full-scale output of ±10 volts.

Access to three outputs is provided. From one output, the filtered, multiplexed signal is fed to the analog-digital converter. The other two outputs provide a continuous signal, which may be used for auxiliary analog recording or monitoring with oscilloscopes, tape recorders, or oscillographs.

Analog-to-digital converter

Connected to the output of the multiplexed amplifiers is the analog-to-digital converter, commonly called the digitizer, which accepts analog voltages within a range of from +10 to −10 volts and generates parallel digital output signals. The output consists of 12 binary digits, one of which is used as the sign bit. The remaining 11 are used to represent the magnitude of the analog voltage. The conversion rate is about 70,000 12-bit words per second.

Mounted just below the digitizer in the center cabinet is a visual display of the data, where the binary information from the digitizer is shown in octal form for any selected channel. The visual display is of help when the circuits are being balanced and calibrated, as separate voltmeter or ohmmeter readings are unnecessary.

ata processing unit—computer

The heart and brain of the instrumentation stem is the SDS 910 computer. It accepts e raw data from the digitizer, processes d stores information, enables data to be inted out, and controls the sequencer and, ereby, the sampling rate. It has a basic re memory of 4,096 words, which could be panded by adding additional memory odules. The word size is 24 binary digits. our hardware priority interrupts are pro- ded. The language used for programing is symbolic language, but conversion routines e available so that FORTRAN II may also used. The computer has buffered input- tput capabilities at rates in excess of 60,000 aracters per second, simultaneous with mputation.

Operations in the system are controlled a clock, which is a 50 kHz. tuning fork cillator divided down by the timing gen- ator to the appropriate frequencies used roughout the system. The timing generator nits a one-pulse-per-second interrupt signal r the computer to update the elapsed time unters.

Input-output device

Communication with the computer is achieved by a teletype machine with paper tape reader and punch placed on line. The maximum speeds of the teletype reader and punch are 10 characters per second each. Usually the program is read in with the tape reader, and specific instructions, for which provisions have been made in the program, are typed in on the teletype. Manual access to the computer is also possible through control switches on the face of the computer panel.

Computer Program

Development of the system also included the writing of a program for the gathering of strain history data on highway bridges. This program was subsequently revised and is described in detail in the succeeding paragraphs.

Program objectives

The principal objective of the program was to count the number of times that a maximum strain range occurs. A *maximum strain range* is defined as the maximum difference in strain that is caused by the passage of a single vehicle. Ten channels are monitored and a counting table for the various strain ranges is maintained for each channel.

The strain ranges are computed by establishing strain levels, as shown in figure 3. The strains on the bridge are presented to the system as analog voltages, usually in the millivolt range. They are conditioned and amplified to a maximum range of ±10 volts, full scale, and are subsequently converted to digital notation of ±2047, full scale.

The strain levels are selected by the operator and represent a percentage of full scale. The program allows for 10 strain levels per channel. When a peak, such as point A in figure 3, is detected, a search is made to determine in which level the point is located. The same procedure is then followed for a valley, such as point B. The maximum strain range is computed as the difference of the strain levels that contain the peak and valley—not the actual values of peak and valley.

To avoid the undesirable counting of peaks and valleys caused by passenger cars and other light vehicles and those caused by small oscillations of the bridge itself, a *test level*, shown as the dashed line in figure 3, is established by the operator. A peak is not recorded

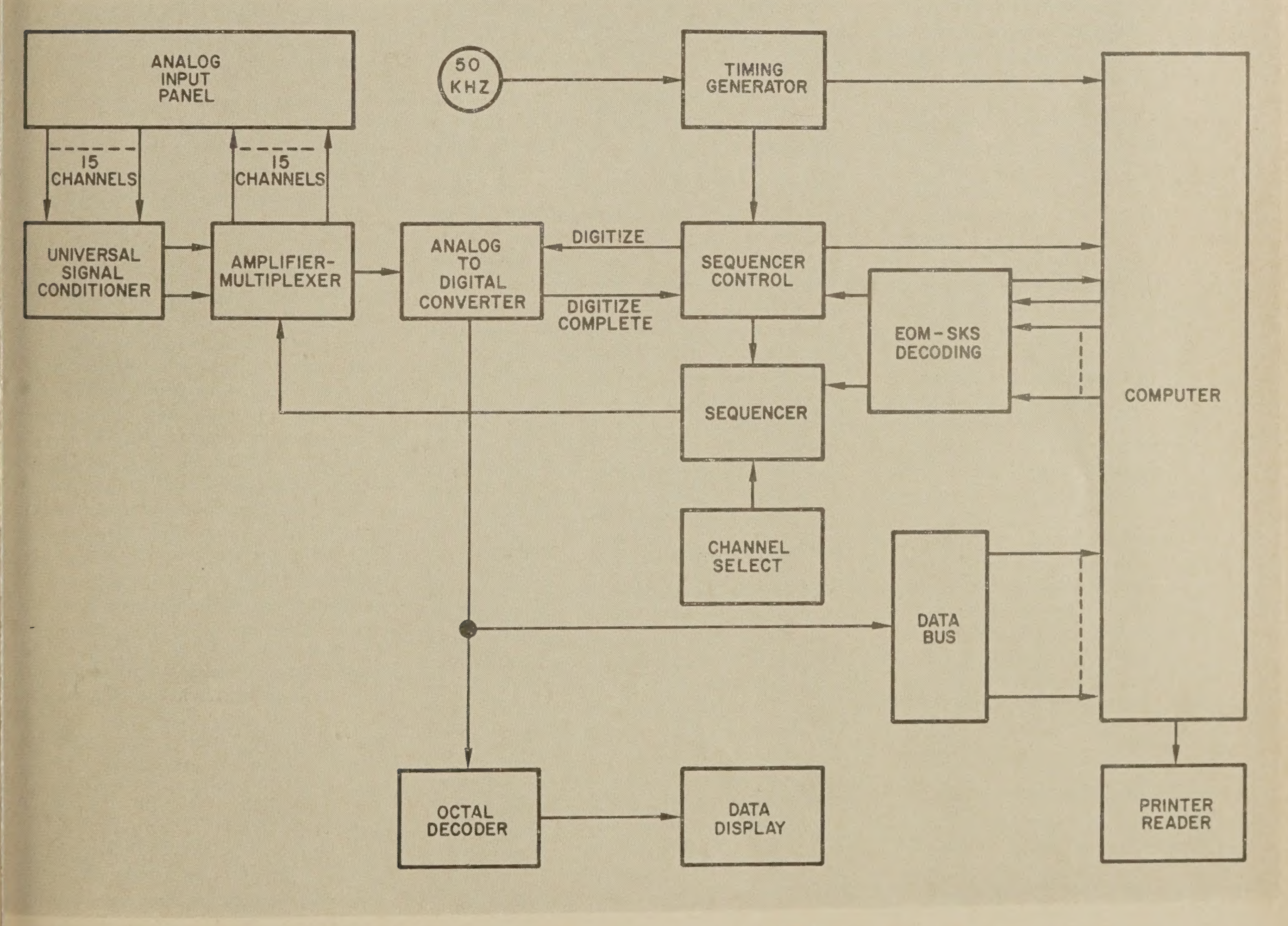


Figure 2.—Block diagram, data acquisition system.

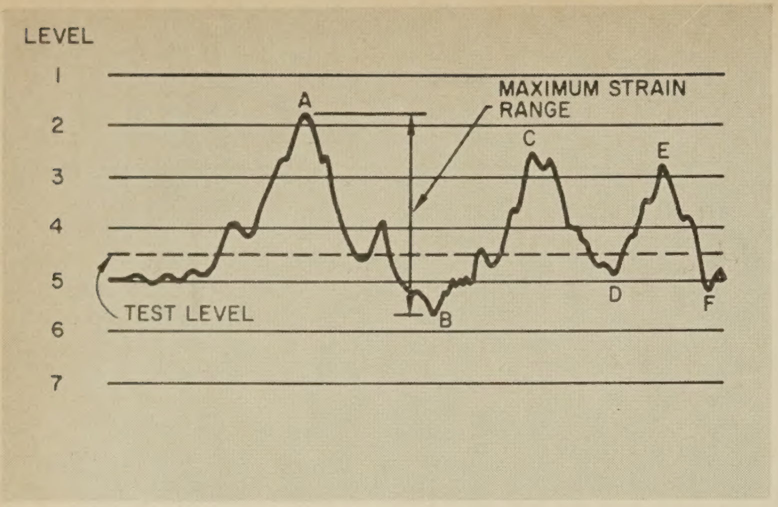


Figure 3.—Maximum strain range.

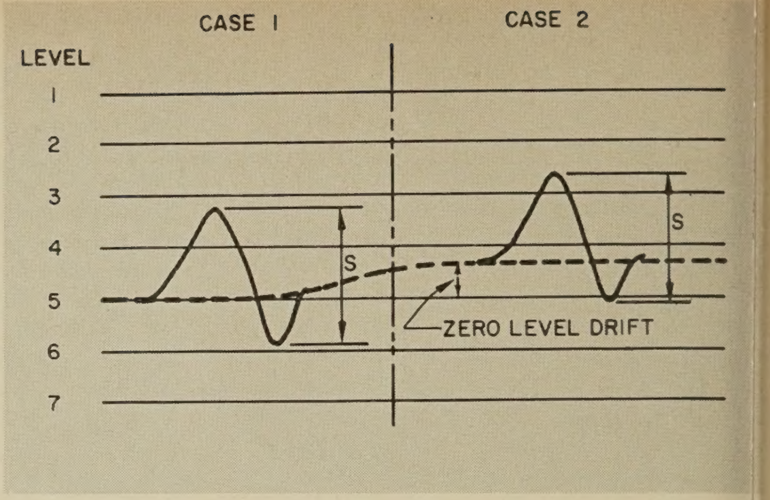


Figure 4.—Zero level drifting.

unless the occurrence exceeds this test level. To be certain that the maximum strain range is obtained, a corresponding valley is not recorded until the strain drops below the zero level.

Another objective of the program was to separate live-load strains from those caused by environmental conditions. It was intended to continuously monitor strains at any particular bridge for periods of several days. During this time the bridge will experience numerous cycles of strain caused by temperature changes. This strain is separated from live-load strains by a zero level adjustment, which is periodically performed.

When the operator determines the strain levels to be used for each channel, he also determines which of these levels will be the zero strain level. During the time that the system is sampling the 10 channels, some drifting of this zero level occurs, as shown in figure 4. To correct for this drifting, each channel is periodically scanned for its new zero level, which is found by sampling until a relatively flat curve—continuous samples of very close magnitude—is detected for a continuous, specified, length of time. This time can be varied by the operator to fit a particular traffic situation. The average value of this flat curve is computed as the new zero level. All strain levels are then shifted by the amount that the zero level shifts from its previous value.

The frequency at which the zero level adjustment is to be performed is determined by the operator when he initiates the program. Five zero level adjustments are allowed during each recording interval. As the digital value of each zero level is printed out at the end of each recording interval, the slowly varying strains in a member caused by temperature changes can be reconstructed merely by converting the values to strains.

Data acquisition and processing

The program is designed to accept and process 15 channels of low-level transducer information. Ten of these channels, primary channels, are monitored for associated peak and valley counts as described earlier. The remaining five channels, secondary channels, may be used for monitoring other transducer signals, such as thermocouples.

The samples from the 10 primary channels are processed first, beginning with channel No. 1. After processing the first sample, the program advances the sequencer to the next channel. Each sample is checked to see whether it is larger than the test level. (See fig. 4.) If it is, it is checked to see whether it is a peak. If so, the level containing the peak is recorded. If another, larger peak occurs before the signal drops below the zero level, the larger peak is recorded in place of the smaller one.

When the signal drops below the zero level, the program starts searching for a valley. It continues to search for valleys until the signal again becomes larger than the test level. At this time, the most negative strain level containing the valley is recorded, and a count is made in the appropriate storage location corresponding to the peak and valley just recorded. This process continues throughout the recording interval.

When the real-time clock detects that it is time for a zero level adjustment, the system processes the zero level adjustment program, which uses the interrupt system to obtain the samples. All 15 channels are sampled in this operation. The samples from each of the channels are accumulated, and the average value computed and saved for output. The zero level adjustment is then performed.

Program summary

The program presents a reasonably accurate method for detecting the maximum strain ranges that occur in a stressed member under cyclic live loading. For each such occurrence, a count is made in the proper matrix accumulator. The counts are accumulated for each channel separately for a predetermined length of time.

The accuracy of the maximum strain ranges recorded depends on several factors. One is the frequency of the cyclic live loading. The maximum sampling rate for each channel is set at 200 samples per second. As the response frequency increases, accuracy decreases. Good accuracy can be obtained with frequencies below 20 cycles per second, and excellent accuracy can be obtained at frequencies below 10 cycles per second.

Another factor that influences the accuracy of the maximum strain ranges is drifting of the zero strain level. The program, by performing a zero level adjustment at predetermined intervals, presents a method for separating and determining strains in members caused by environmental conditions. But between intervals, a peak and corresponding valley could be recorded in different levels, owing to the shift in the zero level by the two cases. As shown in figure 4, this could also affect the maximum strain range. In both cases S is the same, and in both cases the valley will be recorded in level 6, but in case 1, the peak will be recorded in level 4, and in case 2, in level 3. Therefore, the maximum strain range will be recorded differently in each case.

To minimize this difficulty, the zero level adjustment should be performed frequently, and the expected rate of change of the zero level must be considered when setting the length of recording time.

The accuracy of the maximum strain range is also affected by the magnitude of the difference in the strain levels that are chosen. The smaller the difference, the greater will be the accuracy.

One other source of measurement error could stem from the use of the test level, which was established to eliminate small oscillations of the bridge. The program does not search for a peak until the signal exceeds the test level. It searches for a valley only when the signal drops below the zero level. Consequently, erroneous peak and valley associations could occur infrequently. Referring to figure 3, peak C would be associated with valley F, since valley D did not drop below the zero level. Such a situation could occur when one vehicle is following closely behind another.

All these sources of error are considered minor, or can be corrected by a careful adaptation of the sampling scheme to a specific local traffic and environmental condition.

The data acquisition system and the computer program were tested on an actual bridge in a pilot study to determine just how

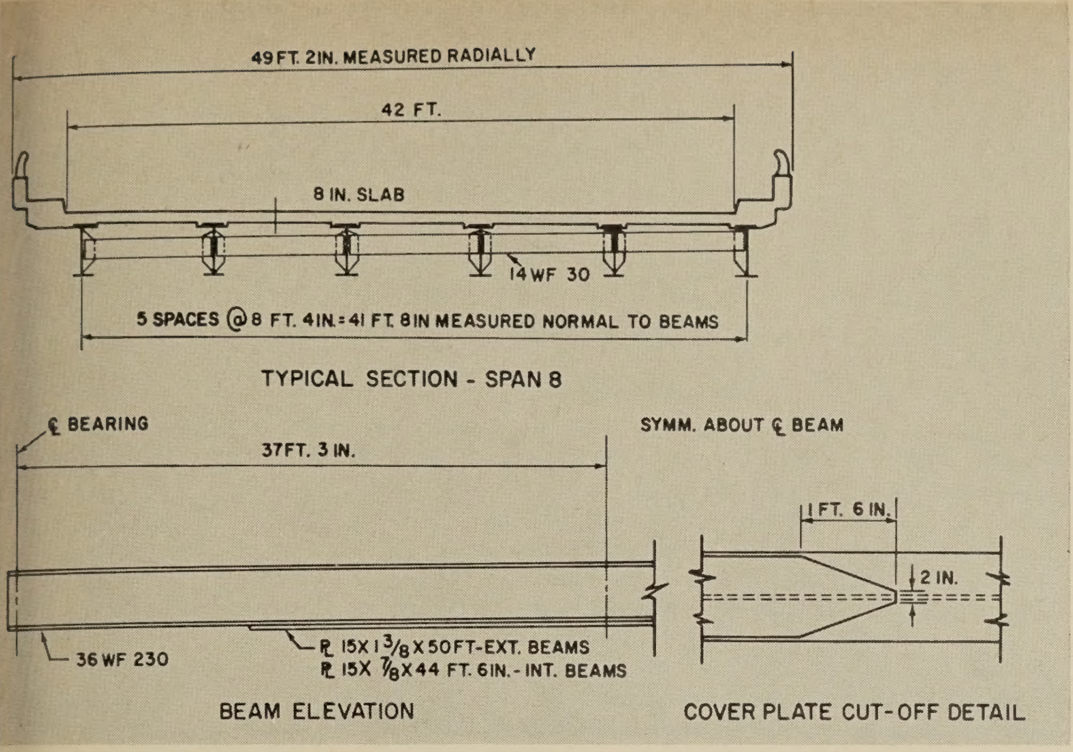

49 FT. 2 IN. MEASURED RADIALLY
42 FT.
8 IN. SLAB
14 WF 30
5 SPACES @ 8 FT. 4 IN.= 41 FT. 8 IN MEASURED NORMAL TO BEAMS
TYPICAL SECTION - SPAN 8
℄ BEARING
37 FT. 3 IN.
SYMM. ABOUT ℄ BEAM
1 FT. 6 IN.
2 IN.
36 WF 230
℞ 15X 1 3/8 X 50 FT-EXT. BEAMS
℞ 15X 7/8 X 44 FT. 6 IN.- INT. BEAMS
BEAM ELEVATION
COVER PLATE CUT-OFF DETAIL

Figure 5.—Dumfries bridge, northernmost span.

ell they perform the functions for which ley were designed. The conduct of the test, le results obtained, and an evaluation of the ystem, make up the remainder of this article.

Dumfries Field Study

The bridge chosen for the pilot study was steel girder structure with partial-length, elded cover plates. It was a simple, four-pan bridge of composite design located in le northbound lane of Interstate Highway 5 near Dumfries, Va. This site was chosen rimarily because of the heavy truck traffic accommodates, and also because of its earness to a truck weighing station. Views f the structure are shown at the beginning of is article. The bridge has a slight skew ngle, left forward at 3°46′.

Of the four spans, only the northernmost pan was instrumented and tested. Structural etails of this span are shown in figure 5. en strain gages were placed on the bottom anges of five of the girders. The sixth girder, eing the outside girder on the left side of the ridge, would have received little stress om truck traffic and was not instrumented.

On the instrumented girders, one gage was laced in the center of the bottom of the over plate at midspan on a skewed section. nother was placed on the bottom of the irder 4 inches off the south end of the over plate.

During the period of testing, strains were ontinuously monitored, the sampling intervals ranging from ½ to 2 hours. At the end of ach sampling interval, the accumulated ata was printed out as strain level peaks nd associated valleys. From the summation f these periodic outputs, the stress range istribution for each instrumented member f the bridge was determined, as shown in gure 6. Values from beam 4 are not shown because the gages on the beam malfunctioned during part of the sampling periods.

The data in figure 6 were recorded from May to November 1967 during scattered sampling intervals. The frequency of occurrence of each stress range, computed from the total number of occurrences at each stress level for each beam, is shown in table 1. Beams were numbered right to left looking north. The maximum stress range, between 3,300 and 3,700 p.s.i. occurred on beam 3. However, considering the total number of stress range occurrences above 1,650 p.s.i., there was a larger number on beam 2 than on any other beam. Midspan stresses are shown in figure 6, but end-of-cover-plate stresses were about the same as these.

The number of occurrences shown in table 1 relates to the number of vehicles that crossed the structure, as follows. In addition to the absolute maximum stress range excursion, certain vehicles produced complementary stress cycles, which were also counted. Some of these complementary cycles could have had greater amplitudes than the absolute maximum induced stress caused by other vehicles. For example, figure 3 could have represented the stress caused by the passage of one vehicle. However, two occurrences would have been recorded—A–B and C–F. By keeping a record of the number of trucks that crossed the bridge during the time that strain ranges were recorded, it was possible to obtain the average number of occurrences per vehicle for each beam. The values for this bridge (table 1), ranged from 1.7 for beam 5 to 3.0 for beam 3. Beam 3 had not only the highest recorded stress range, but also the highest ratio of occurrences per event.

Truck type and weight distribution

A State-operated weighing station was located approximately 2 miles from the Dumfries bridge. This station, in continuous operation throughout the year, made it possible to obtain accurate truck counts. The monthly and yearly truck-traffic totals for 1966 and 1967 are shown in table 2. The

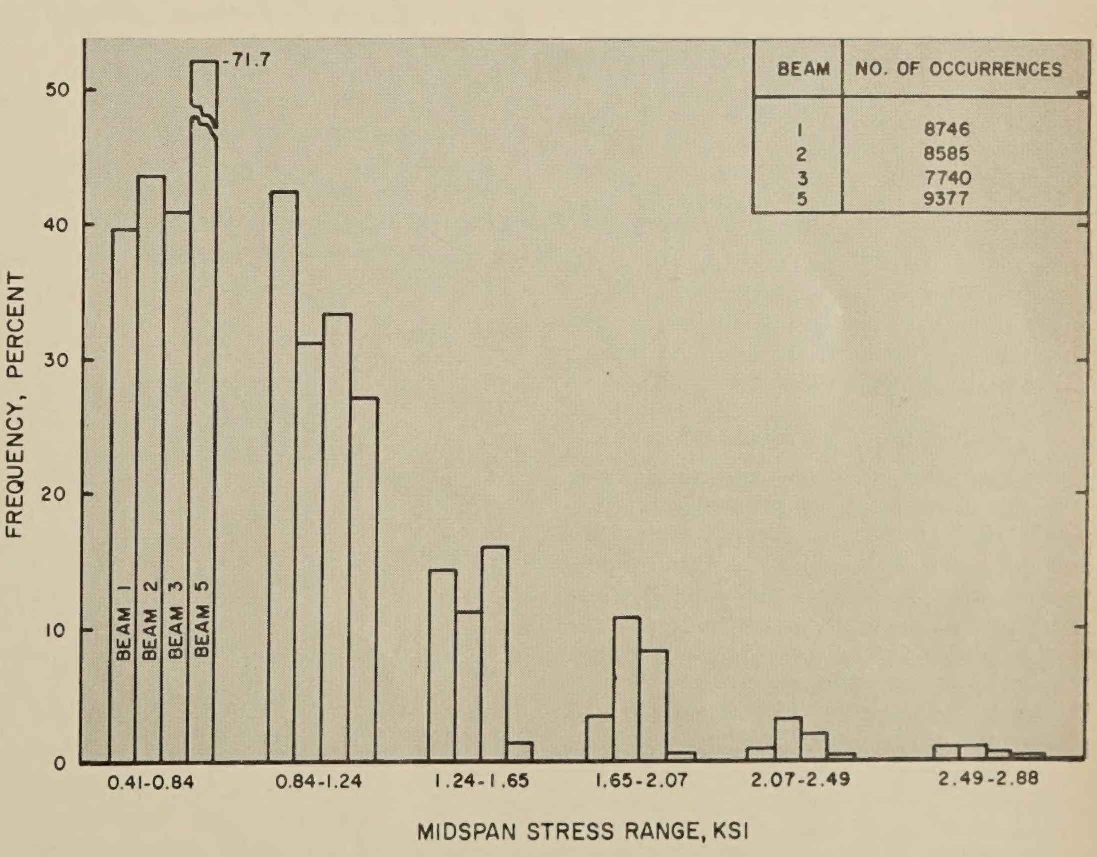


Figure 6.—Frequency distribution of stress ranges at midspan.

highest volume of truck traffic occurred during the summer months. The Virginia truck law allows a maximum gross weight of 70,000 pounds, but there is an unofficial enforcement tolerance of 5 percent, which would raise the maximum gross weight to 73,500 pounds. The truck traffic included two 5-axle vehicles, distributed as shown in figure 7.

Records of truck weights were collected at the weighing station during scattered intervals while strains were monitored. From these data, the gross truck weight distribution was determined as shown in figure 8. The greater percentage of truck traffic weighed between 20 and 30 kips, with only 5.6 percent above 70 kips. The average gross weight of each axle group is given in figure 7.

Life expectancy

An attempt was made to predict the life expectancy, or fatigue life, of the Dumfries bridge. The fatigue curve used was developed from the recommendations of the joint ASCE–AASHO Committee on Flexural Members (*1*).[2] For rolled beams with cover plates using ASTM A36 steel, the Committee recommended a stress range value of 9,000 p.s.i. for fracture at 2×10^6 cycles. In *House Document 354* (*2*), it is assumed that the stress range value at 200×10^6 cycles is equal to one-third the stress range value at 2×10^6 cycles. If the same assumption is used for the study reported here, the stress range at 200×10^6 cycles is 3,000 p.s.i. Based on the assumption that the stress range and cycles to failure have a linear log-log relation, figue 9 was constructed. This curve, along with Miner's hypothesis of cumulative damage, can be used to predict the fatigue life of a bridge, if the stress ranges to which it will be subjected are known. Other researchers (*3*, *4*) also have developed fatigue curves similar to the one shown in figure 9.

The stress ranges recorded on the Dumfries bridge were not large enough to make a reliable estimate of its fatigue life. The highest stress range recorded, approximately 3,500 p.s.i. on beam 3 at midspan, had an occurrence frequency of less than 0.01 percent. Slightly lower stresses were recorded near the ends of the cover plates. The frequency of occurrence of all stresses on beam 3, which fell on the curve of figure 9, was only 0.06 percent. It is therefore impossible to predict the fatigue life of the bridge from these data.

Summary and Discussion

Evaluation of the data acquisition system indicates that the concept of combining transducer conditioning, amplifying, monitoring, and processing instrumentation into one system controlled by a digital computer is a very good one. The system, as assembled, is certainly able to monitor, digitize, and process the strains produced by ordinary traffic, without any loss of essential information and with a minimum of manpower in attendance.

Having a computer controlled data acquisition system allows for great flexibility in gathering and processing data. One guiding principle during the development of the program was simplicity. As several aspects of the fatigue problem on highway bridges are not known or understood, it was decided to concentrate only on the gathering of major stress ranges. All kinds of minor vibrations and fluctuations in stress could also have been recorded; but these have been shown to have no bearing on the life expectancy of highway structures, and the choice was made to ignore them.

Laboratory fatigue studies together with the development of an applicable cumulative damage fatigue theory will have to accompany loading history studies in the field.

Those aspects of the program relating to the zero level adjustment procedure are believed to be sound, and any errors caused by drifting that are not corrected by the zero level adjustment are very small.

The Dumfries field test was properly named a pilot study, as it was not a full fledged investigation into the loading history of a highway bridge. Initially, the bridge served mainly to check out the new instrumentation, but the volume of data gathered far surpasses that of other investigators who used oscillograph instrumentation to record the passage of trucks in similar studies.

For instance, in a recent Michigan study (*3*) involving eight bridges, only some 3,000 events were recorded for a single bridge. In a similar investigation in Maryland during the summer of 1967, only about 1,000 events were recorded. In the Dumfries study, the average for each of the several beams instrumented was about 9,000 stress events. Many more were recorded in the early parts of the study, but these were not reported here because the initial computer program did not clearly isolate stress ranges, recording only at which level a maximum peak occurred.

Before any conclusions can be based on the results, not only in this study but in all other similar ones, it must be determined whether the sampling of data was representative of the effects produced by the truck traffic—in 1 year for instance.

The results reported here were recorded during the following sampling periods:

May 1—½ hour, midday
2—4 hours, midday
3—3 hours, midday
July 5—1 hour, midday
6—2 hours, midday
7—4 hours, midday
October 26—4 hours, midday
27–30—64 hours, continuous run over 1 weekend
31—4 hours, midday

There was no sampling in winter or early spring, and except for the continuous weekend run, the sampling periods fell in the middle of the day, some time between 10 o'clock in the morning and 4 o'clock in the afternoon. However, it is again mentioned that other researchers in similar studies considered their sampling time, although much shorter, to be representative.

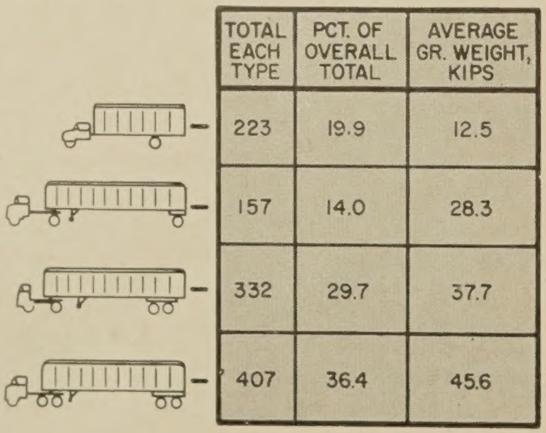
The results of the field study, as shown in the stress range histogram, figure 7, compare favorably with the results of the recent similar studies in Michigan and Maryland. An interesting comparison among nine slab and steel girder bridges can be made from the data presented in table 3. In general, the maximum measured stress ranges are well below the calculated design live-load stresses, including impact. In fact, based on the 161

Table 1.—Number of stress range occurrences, midspan gages

Beam	Number of occurrences										
	Stress, range, p.s.i.									Total	Per vehicle
	3,700	3,300	2,880	2,490	2,070	1,650	1,240	840	410		
1		---------	---------	3	39	287	1,255	3,687	3,475	8,746	2.1
2		---------	2	36	276	906	935	2,674	3,756	8,585	1.9
3		1	4	24	150	615	1,236	2,579	3,131	7,740	3.0
5		---------	---------	1	2	12	91	2,554	6,717	9,377	1.7

Table 2.—Number of trucks weighed, Dumfries weighing station, route I–95, northbound

Month	1966	1967
January	41,459	36,678
February	42,149	42,823
March	49,028	50,840
April	53,841	43,954
May	51,846	50,497
June	58,322	60,176
July	50,629	47,776
August	58,605	60,123
September	49,385	49,265
October	57,406	54,346
November	44,129	51,886
December	26,409	45,355
Total	583,208	593,719



	TOTAL EACH TYPE	PCT. OF OVERALL TOTAL	AVERAGE GR. WEIGHT, KIPS
	223	19.9	12.5
	157	14.0	28.3
	332	29.7	37.7
	407	36.4	45.6

Figure 7.—Truck type and weight distribution.

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[2] Italic numbers in parentheses refer to the references listed on page 189.

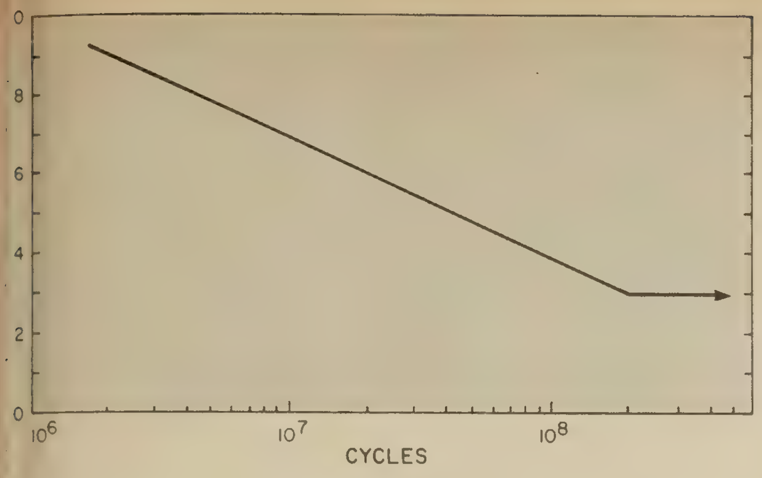


Figure 8.—Frequency distribution of truck gross weights.

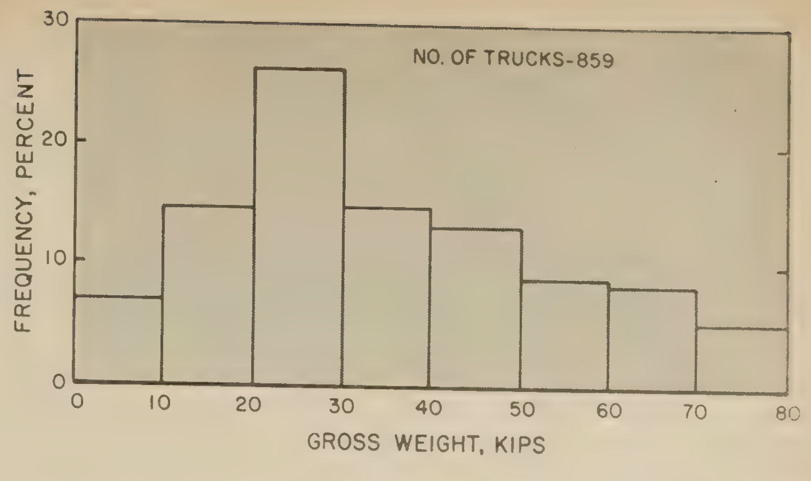


Figure 9.—Assumed S–N curve for beam with welded partial-length cover plates.

..SHO Bridge Specifications, the Dumfries ximum measured stress range was only 45 cent of the design live-load stress for an erior beam. This maximum measured stress ge of 3,500 p.s.i. occurred only once in re than 8,000 events, and if the value of roximately 2,700 p.s.i. (figure 7) had been d, an even lower ratio would have resulted.

The Dumfries bridge is a 3-lane bridge h a 2½-foot sidewalk on both sides of the dge. It is located on a rural portion of a Interstate highway and crosses a creek well as a small road; it is therefore at a point in the vertical alinement of the d. During the conduct of the test it was asionally noted that one truck attempting passing maneuver would be exactly even h another in the centerlane, and both cks crossed the test span at the same tant. However, most of the trucks crossed test span singly, traveling near or above speed limit of 65 m.p.h. Never were all 3 es of the bridge observed to be loaded h trucks simultaneously. As the live-load ign calculations are based on 2 or more es being loaded simultaneously and on tain load distribution factors, the low orded stress ranges can be explained in t by the absence of such simultaneous ding. This situation, of course, is peculiar this structure only. Closer agreement ween calculated and recorded live-load esses would be expected on longer span, ane bridges.

Because of the low recorded stresses, it was possible to obtain a meaningful forecast of possible fatigue damage, as pointed out lier.

ver-plate gages

The results from the gages that were placed nches off the south end (traffic is north-und) of the cover plates were very much e the results from the midspan gages, both distribution and magnitude. These results not presented here.

resses caused by temperature

The capability of the instrumentation, with properly programed instructions, to arately monitor and record live-load strains, as well as strains caused by temperature differentials, and perhaps moisture differentials, was described earlier. The desirability for having this capability is twofold: (1) It assures that the live-load strains are not clouded by any drifting of a baseline, and (2) it allows a monitoring of such drifting. In no way can one be certain that all the recorded drift is due to actual strains in the bridge, unless such a condition has been independently verified by another type of strain measurement.

In the Dumfries field test, extreme care was exercised to eliminate as many errors as possible. Amplifier drifting and the drifting of other instrumentation was shown to be negligible when a very stable load cell was used. Temperature compensation in the dummy gages was provided, and the dummy gages were placed as near as possible to the active gages. The one untested source of error might be due to variations in lead wire behavior during the course of a day.

It would be improper to publish any of the recorded long-term strains, as they might contain errors; but it can be said that the magnitude of the long-term strains appears to be two to three times as large as the maximum recorded live-load strain. Also, there seems to be a great variation in long-term strains between gage positions, such as at midspan and at end of cover plate on the same beam, as well as between interior and exterior beams.

Conclusions

Based on experience gained while working with the data acquisition system and on the results of the Dumfries field tests, the following conclusions seem in order:

- The data acquisition system can adequately accomplish the task it was designed to perform—to monitor and digitize strain history data on highway bridges.
- The results from the Dumfries field tests, in the form of measured stress ranges, were always less than half the calculated design live load with impact stress, and usually were much smaller than half.
- The measured stress ranges obtained from the Dumfries bridge are somewhat lower

(Continued on p. 189)

Table 3.—Stress ranges of different bridges

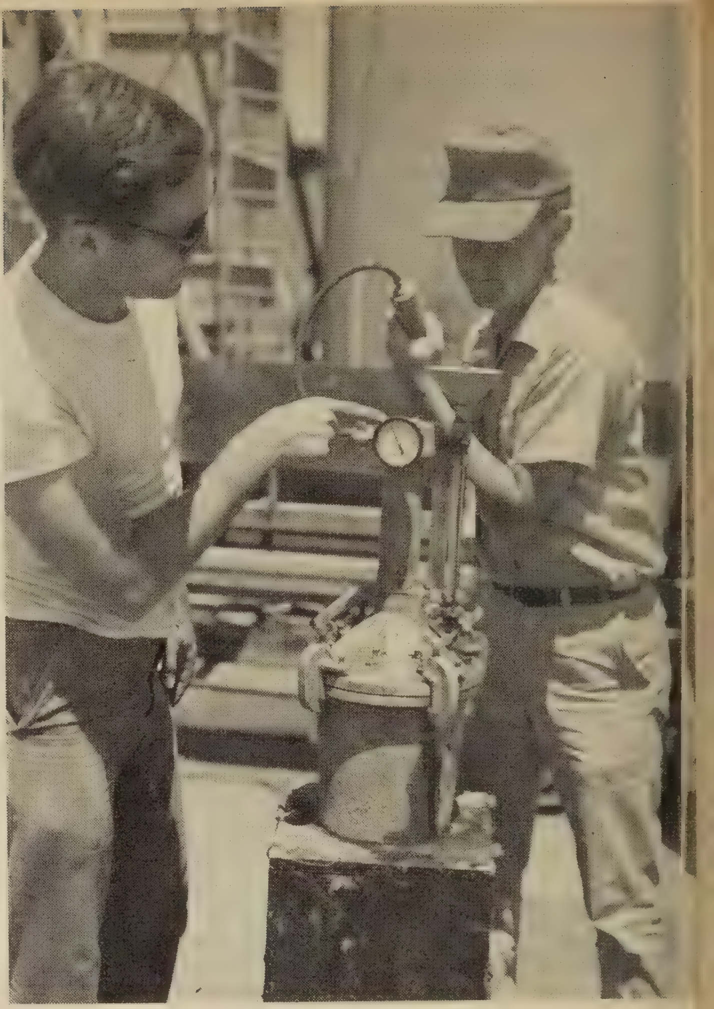
Bridge	Maximum stress range	Occurrence	$\frac{\text{Maximum measured}}{\text{Design LL+I}}$
	p.s.i.	*Percent*	
Michigan 1	6,300	0.1	$\frac{6300}{8040}=0.78$
Michigan 2	5,100	0.1	$\frac{5100}{7730}=0.66$
Michigan 3	5,100	0.3	$\frac{5100}{8500}=0.60$
Michigan 5	4,500	0.4	$\frac{4500}{7820}=0.58$
Michigan 6	3,900	0.1	$\frac{3900}{8950}=0.44$
Michigan 7	4,500	0.1	$\frac{4500}{7030}=0.64$
Michigan 8	6,300	0.2	$\frac{6300}{7000}=0.90$
Maryland U.S. 1	5,000	0.3	$\frac{5000}{10400}=0.48$
Dumfries	3,500	[1] 0.01	$\frac{3500}{7740}=0.45$

[1] Occurred once out of more than 8,000.

Quality Assurance in Highway Construction

Part 3—
Quality Assurance of Portland Cement Concrete

Reported by **WESLEY M. BAKER**, Highway Research Engineer, and **THURMUL F. McMAHON**, Principal Quality-Assurance Research Engineer, Materials Division



Pressure meter test for determining air content in portland cement concrete.

This is the third part of an interpretative summary of the progress in Public Roads research program for the statistical approach to quality assurance in highway construction. Part 1.—Introduction and Concepts, and Part 2.—Quality Assurance of Embankments and Base Courses, were presented in previous issues of PUBLIC ROADS. The remaining parts, to be presented in succeeding issues, are 4.—Variations of Bituminous Construction, 5.—Summary of Research for Quality Assurance of Aggregate, and 6.—Control Charts.

BY THE OFFICE OF
RESEARCH AND DEVELOPMENT
BUREAU OF PUBLIC ROADS

Introduction

EVER since the development of portland cement concrete for use in the construction of highway pavements and structures, highway engineers have been concerned with the quality of the concrete and of its constituents. From years of experience, methods have been developed to control the quality of concrete and measure its acceptability as a quality material. But responsibility for the quality of portland cement concrete and causes of its failure are still confusing issues.

Research has provided an insight into the causes of variations that have always existed in the results of concrete tests. It has also provided some knowledge of their magnitude as regards blending, mixing, sampling, and testing, especially under laboratory conditions. In early research it was recognized that statistical concepts afforded a useful tool to analyze the data and establish the causes of variation. In fact, as early as the 1940's, Mr. Alfred M. Fruedenthal recommended that statistical concepts be used to revise portland cement concrete specifications (*1*).[1]

A committee on quality control of concrete in the field, appointed in England at the beginning of the 1950's, reviewed all aspects of concrete production and recommended methods of improving quality and testing techniques. The committee's greatest achievement was the adoption of statistical concepts to better understand the nature of variation in concrete production. The normal distribution was shown to be applicable in the concrete industry, particularly for the cylinder strength distribution, and acceptance criteria were established at a 95-percent confidence level.

The first official action in the United States to adapt the statistical approach to quality control of concrete was taken in 1955 by the American Concrete Institute. Criteria were established and rational specifications or structural concrete were recommended.

In this report on the application of statistical quality control methods to the production of concrete by the highway industry (*2*), Mr. Edward A. Abdun-Nur advocated extensive use of statistical concepts in specification writing and acceptance sampling. He considered a realistic picture of concrete produced under normal control to be one in which the coefficient of variation of 28-day strength is 20–25 percent, and defined a good concrete as one with a 15-percent coefficient of variation.

Early work in quality control of portland cement concrete demonstrated the advantages of using statistical concepts to specify, control, and accept concrete; however, more information was needed to make optimum use of them. The Office of Research and Development, Bureau of Public Roads, has been promoting the gathering of information by the States concerning the quality of the concrete being produced under current speciications and the contributing factors in the ariation of test results. The objective of his research program has not been to determine all factors relating to variations in oncrete production, but to isolate variations wing to materials, sampling, and testing. The esults to date are presented and analyzed n the following paragraphs.

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[1] Italic numbers in parentheses identify the references listed on p. 189.

Variability in Concrete Strengths

Strength is not always the most important haracteristic of concrete quality, but it is he one that is most often measured. It is ssumed to be indicative of the water-cement atio and, accordingly, an indicator of durability. The magnitude of the variability in trength is, therefore, an indicator of the nagnitude of variability of the other haracteristics.

Variability in concrete strengths can be ttributed to two other types of variability: 1) Inherent variability in the materials and rocesses that results from chance causes, which cannot be controlled, and (2) variability rom assignable causes which can be controlled. The attainable quality of concrete in the field s limited not only by the chance causes that ontribute to the variation in quality, but lso by the economic factors entailed in educing the assignable causes.

The measurement of variability from chance r inherent causes is complicated by the nherent variability of each of the ingredients n the mix, which can interact with the rocesses of blending, mixing, and placing, and result in a much larger variability in the concrete itself.

The assignable causes of variability are more numerous and more difficult to isolate, but the production of quality concrete is dependent on the reduction of all variables. However, the isolation and restriction of variables can be carried only as far as economic conditions warrant. Under the present state of knowledge, the ultimate uniformity of concrete production cannot be precisely stated. Extensive research will be necessary to isolate variables and to determine the extent to which variation can be reduced. Current information can be used only to show that variation does exist, and that sampling and testing often contribute as much of the variability as do the ingredients and processes used in construction.

Mr. H. H. Newlon, in a paper (*3*) described and discussed numerous variables affecting concrete quality. The data concerning concrete variability, presented in table 1, was based on a similar tabulation from his paper. Although such information is interesting and may be used to design specification limits, it is of little worth to the overall problem of reducing variability in concrete construction. The basic need is to isolate the common factors affecting variability. Research aimed at this purpose has been underway for the past 4 years.

Data based on a West Virginia research report (*4*) are presented in table 2. These data are illustrated in figure 1, which depicts the relations among the materials, sampling, and testing standard deviations. As standard deviations are not additive, the sum of the standard deviation shown does not equal the standard deviation of the concrete strength. A significant indication from these data is that the combined testing and sampling variations are usually greater than the material variation. It is also significant that the materials deviations consist of the material and process variations, whereas the sampling and testing deviations are caused by the measurement process. Figure 1 indicates that sampling did not contribute significantly to the variation of test results on these projects.

An analysis of historical data on compressive strengths of concrete cylinders, presented in table 3, was based on a report by the State Road Department of Florida (*5*). Two types of concrete, class *A* and class *NS* were analyzed in the report. Routine control is normally exercised over class *A* concrete, whereas class *NS* concrete is spot checked only occasionally.

Based on the same Florida report the mean strength, standard deviation, and coefficient of variation for the concrete, shown in table 3, were compared by source in table 4. Researchers have shown that the standard deviations of strength test results usually increase with the mean or average strength of the concrete; therefore, the best comparison of these data may be shown by the coefficient of variation. As expected, the coefficients of variation for class *NS* concrete were greater than those for class *A*.

The strength data presented—typical of nearly all strength data received—expressed large standard deviations with average strengths well above the usual minimum of 3,000 p.s.i. For example, if the class *A* concrete of project 4, table 3, were analyzed using normal distribution methods, there would be less than 1 percent chance that a test would result in a compressive strength of less than 3,988 p.s.i. With the same standard deviation, the mean could be as low as 4,698 p.s.i. before a 1-percent chance of being below 3,000 p.s.i. was exceeded. If the standard deviation could be reduced, the mean might be lowered further without risking noncompliance. However, as pointed out previously, the durability

Table 1.—Average deviation of concrete strengths

Agency	Concrete type	Data source	Average standard deviation[1] ($\bar{\sigma}$)
			p.s.i.
Bureau of Public Roads	Paving	Research	585
Do	do	Historical	473
Do	Structural	do	576
Virginia Department of Highways	do	Special	467
Do	Paving	Cores	663
Ontario Department of Highways	do	Routine	494

[1] The average standard deviation for all data presented.

Table 2.—Portland cement concrete variations

Project No.	28-day compressive strength variations								
	Mean ($\bar{X}$)	Overall standard deviation (σ_o)	Overall coefficient of variation (v_o)	Standard deviation, testing (σ_t)	Coefficient of variation, testing (v_t)	Standard deviation, sampling (σ_s)	Coefficient of variation, sampling (v_s)	Standard deviation, materials (σ_a)	Coefficient of variation, materials (v_a)
Structural concrete									
	p.s.i.	*p.s.i.*	*Pct.*	*p.s.i.*	*Pct.*	*p.s.i.*	*Pct.*	*p.s.i.*	*Pct.*
1	4,235	435	10.0	170	4.0	236	5.6	310	7.2
2	4,420	482	10.9	323	7.3	39	0	360	8.1
Paving concrete									
1	4,675	545	11.7	377	8.1	91	0	386	8.3
2	3,755	420	11.2	322	8.5	42	0	270	7.1
3	3,720	575	15.5	318	8.5		0	495	13.3
4	4,760	467	9.8	200	4.2	34	0	420	8.8
5	4,688	733	16.5	585	12.5		0	545	11.7

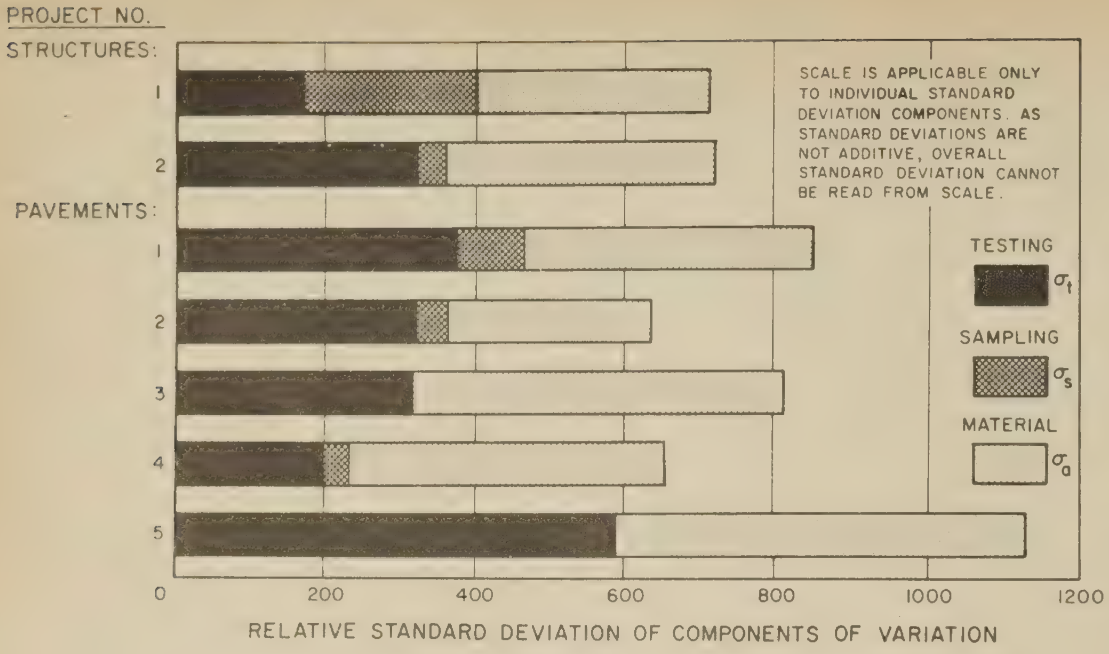

Figure 1.—Portland cement concrete—standard deviation, compressive strength.

of the concrete may be the controlling factor in reducing strength through lowering the design cement content. If it is assumed that strength is an indicator of water-cement ratio, it is possible that more uniform strength will also result in a more uniformly durable concrete. This in turn may allow a reduction of the design cement content.

If durability should be the controlling factor and if it should have a relation with strength, a test for durability should be developed. This test would eventually replace the strength test in measuring concrete quality.

Variability of Plastic-Concrete Air Content

Air content of portland cement concrete is one of the most important factors in the durability of pavements and bridge decks. Not only is it important that the air content be sufficient to prevent damage from freeze and thaw cycles and low enough to preserve strength, but it is also important that the air be distributed uniformly throughout the mix.

Several States have studied variations in air content and evaluated the performance of different test methods. The data shown in table 5, submitted by the State of New York, are representative of this research. The tests proved that, in the State of New York, truck mix concrete was more variable with respect to air than was paver mix or central mix concrete. The tests also showed that the air content measured by the Chace meter was considerably higher than that measured by the pressure meter. As tests with the Chace meter are faster than those with the pressure meter, considerable interest exists in determining the number of Chace meter tests that would give an average that has the same degree of precision as an average based on a lesser number of pressure meter tests.

The equivalency of the Chace and pressur meters can be determined by a compariso of the sampling and testing variance of eac in the following manner:

$$\frac{\sigma_{st}^2(C)}{n}=\frac{\sigma_{st}^2(P)}{1}$$

In project 2, table 5, this results in tl following equivalency:

$$\frac{.354}{n}=\frac{.166}{1}\ \therefore n=2.1$$

or

2 Chace tests = 1 pressure test

Equivalencies computed for the oth projects shown in table 5 ranged from on to 20; six of the 10 were below four, indicatin that averages based on four Chace met tests may be suitable for control purpos However, the wide variation in these resul indicate that the actual equivalency of t two tests depends somewhat on the operato

Table 3.—Historical concrete strength data

	Samples	Mean ($\overline{X}$)	Overall standard deviation (σ_o)	Coefficient of variation (v)	Testing error (σ_t)
28-day cylinders—class *A* concrete					
	Number	*p.s.i.*	*p.s.i.*	*Pct.*	*p.s.i.*
Project number:					
1 [1]	536	4, 524	396	8. 8	175
2	292	4, 881	540	11. 1	207
3	96	5, 686	544	9. 6	185
4	192	5, 527	566	10. 2	155
5	196	5. 098	577	11. 3	[2] 48
6	112	4, 826	608	12. 6	196
7	258	5, 469	667	12. 2	150
[illegible]	320	5, 244	674	12. 9	158
9	232	5, 289	711	13. 4	192
10	176	4, 927	725	14. 7	210
11	126	5, 067	732	14. 4	162
Average [3]	230	5, 140	613	13. 1	179
Range	224	860	192	5. 1	60
28-day cylinders—*NS* concrete					
Project number:					
1 [1]	50	4, 021	[1] 398	9. 9	115
2	340	3, 555	550	15. 5	93
3	240	4, 006	580	14. 5	134
4	200	3, 474	605	17. 4	122
5	240	3. 781	670	17. 7	96
6	196	4. 192	729	17. 4	160
7	148	4. 213	733	17. 4	92
8	94	4. 239	774	18. 3	87
9	108	3, 657	774	21. 2	116
10	182	3. 674	776	21. 1	113
11	156	4, 110	776	18. 9	70
12	224	4, 179	825	19. 8	127
13	138	3, 941	884	22. 4	161
Average [3]	178	3, 926	698	17. 8	114
Range	246	765	334	7. 9	91

[1] Values not included in range calculations.

[2] Statistical outlier—not included in calculation of range or average.

[3] Averages are not weighted and include all values ex the outlier.

Table 4.—Production source comparison, historical concrete strength data

Source	Concrete class	Samples	Mean strength ($\overline{X}$)	Pooled standard deviation ($\bar{\sigma}$)	Coefficient of variation (
		Number	*p.s.i.*	*p.s.i.*	*Pct.*
1	*A*	480	4, 799	593	12. 4
	NS	218	4, 091	620	15. 2
2	*A*	540	4, 906	672	13. 7
	NS	360	4, 210	660	15. 7
3	*A*	440	5, 054	585	11. 6
	NS	308	4, 031	584	14. 5

Table 5.—Air content of plastic concrete, research data

Project No.	Mixer type and use	Observations	Test method	Testing variance (σ_t^2)	Sampling variance (σ_s)	Material variance (σ_a^2)	Standard deviation (σ)	Mean ($\bar{X}$)
		Number		*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
1	Central mix	216	Pressure	0.043	0.07	0.80	0.95	4.55
	Paving	216	Chace	0.20	0.545	0.65	1.18	6.39
2	Central mix	200	Pressure	0.126	0.04	0.50	0.82	5.91
	Paving	200	Chace	0.22	0.134	0.70	1.026	7.42
3	E-34 paver	200	Pressure	0.067	0.05	0.48	0.77	5.14
	Paving	200	Chace	0.15	0.10	0.33	0.76	7.38
4	Truck mix	200	Pressure	0.16	0.29	1.74	1.48	7.90
	Structural	200	Chace	0.335	0.15	1.38	1.36	10.2
5	Truck mix	204	Pressure	0.035	0.04	1.27	1.16	6.11
	Structural	204	Chace	0.45	0.99	0.48	1.39	8.75
6	Central mix	200	Pressure	0.06	0.105	0.34	0.71	6.18
	Paving	200	Chace	0.256	0.137	0.40	0.89	7.39
7	E-34 paver	200	Pressure	0.08	0.08	0.39	0.74	4.94
	Paving	200	Chace	0.14	2.55	−0.96	1.32	6.41
8	E-34 paver	200	Pressure	0.047	0.04	0.70	0.89	4.82
	Paving	200	Chace	0.26	0.25	1.02	1.24	6.51
9	Truck mix	200	Pressure	0.05	0.135	2.39	1.60	5.80
	Paving	200	Chace	0.43	0.325	1.79	1.60	8.43
10	Truck mix	200	Pressure	0.09	0.136	1.64	1.37	6.07
	Structural	200	Chace	0.28	0.24	1.71	1.49	6.33

xterity; and, consequently, it may be ecessary to establish operator equivalencies provide sufficient confidence for the control air content by the Chace meter in any test. The data on air content presented in table was reported by West Virginia (4). The ta indicate that in measuring air content, ere is good agreement between the Chace d Roll-A-Meter. Calculation of the equivency of the two tests indicates that in est Virginia, four Chace tests will adequately bstitute for one Roll-A-Meter test.

Table 6.—Portland cement concrete pavement air content, research data

Project No.	Observations	Test method	Testing variance (σ_t^2)	Sampling variance (σ_s^2)	Material variance (σ_a^2)	Standard deviation (σ)	Mean ($\bar{X}$)
	Number		*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct*
1	176	Roll-A-Meter	0.102	0.133	1.352	1.21	5.8
	141	Chace	0.334	0.233	0.565	1.02	5.06
2	104	Roll-A-Meter	0.109	0.608	0.000	0.83	5.8
	104	Chace	0.470	0.710	0.000	1.00	5.46
3	200	Roll-A-Meter	0.153	0.362	1.042	1.24	5.1
	196	Chace	0.531	0.769	0.913	1.30	5.0
4	172	Roll-A-Meter	0.126	0.248	0.191	0.73	5.1
	154	Chace	0.271	1.026	0.08	1.09	4.8
5	192	Roll-A-Meter	0.143	0.110	1.229	1.16	5.[illegible]
	198	Chace	0.249	0.148	1.36	1.32	5.54

ariability of Concrete Consistency

The consistency of plastic concrete, as easured by slump cone or Kelly Ball tests, a measurement of the workability of the ix and an indicator of the water content. owever, consistency is no direct measurement of the water content, as air, gradation, d temperature also affect the consistency. he results of these tests therefore are a good dicator of the uniformity of the mix, and late to a combination of these factors rather an to any one of them.

Results of studies by several States of ncrete consistency, as measured by the ump cone, are presented in table 7. The ta indicate little difference in the variability slump among the several methods of ncrete production, although there is conderable difference from project to project. he data also indicate that actual material riation contributes more to the overall riation than do sampling and testing.

Data from reports by West Virginia (4) d California (6) on studies to evaluate the elly Ball test are shown in table 8. These ta and that from other sources indicate at the Kelly Ball test is valid for measuring e consistency of concrete when three adings are averaged to obtain one test lue as required in the standard method.

Table 7.—Variability of concrete consistency, slump cone method

Project No.	Observations	Testing variance (σ_t^2)	Sampling variance (σ_s^2)	Material variance (σ_a^2)	Overall standard deviation (σ_o)	Mean ($\bar{X}$)	Specification limits
			State 1				
	Number	*Inch*	*Inch*	*Inch*	*Inch*	*Inches*	*Inches*
[1] 1	184	0.16	0.04	0.26	0.68	2.44	0.5–3.5
[2] 2	200	0.13	0.02	0.45	0.80	1.5	0.5–3.5
[2] 3	300	0.25	0.09	0.46	0.89	2.76	0.5–3.5
			State 2				
[3] 1	216	0.074	0.00	0.15	0.47	2.04	----------
[4] 2	200	0.06	0.06	0.37	0.70	1.86	
[3] 3	200	0.08	0.025	0.42	0.73	2.34	----------
[5] 4	200	0.027	0.012	0.206	0.495	1.77	----------
[5] 5	204	0.066	0.03	0.305	0.633	2.37	----------
[6] 6	200	0.033	0.034	0.14	0.456	2.12	----------
[7] 7	200	0.084	0.086	0.20	0.609	2.41	----------
8	200	0.158	0.047	0.50	0.844	2.26	----------

[1] Pavement concrete, truck mix.
[2] Pavement concrete, truck mix, slipform.
[3] Pavement concrete, central mix, screw spreader.
[4] Concrete base, central mix, slide spreader.
[5] Structural concrete, truck mix.
[6] Pavement concrete, central mix, slipform.
[7] Pavement concrete E-34 paver.

Aggregate Size Variation

One factor stressed in concrete specifications d in concrete-production control is gradaon of coarse and fine aggregates. Research ows that, within projects and between ojects, there is considerable variation in size distribution of aggregates in the mix. In fact, aggregates-size-distribution specifications are seldom complied with. Part 5 will contain a detailed report on the gradation of concrete aggregates, but table 9 is included here to illustrate the variation. The data from project 1 indicate little variation of either material or of sampling and testing; however, the data from projects 2 and 3 indicate a large material variance, and it is probable that the specification limits were exceeded on many individual tests.

Variability in Pavement Thickness

Pavement life expectancy is based on estimated traffic and pavement design thickness. There is still argument as to whether the design of the pavement should be based on minimum thickness or average thickness; however, as in all structures, stresses are

Table 8.—Methods of measuring consistency of plastic concrete, research data

Project No.	Samples	Test	Mean ($\bar{X}$)	Standard deviation (σ)	Testing variance (σ_t^2)	Sampling variance (σ_s^2)	Material variance (σ_a^2)
		West Virginia					
	Number		*Inches*	*Inches*	*Inches*	*Inches*	*Inches*
1	200	{Slump	2.4	0.8	0.095	0.095	0.52
		{Kelly Ball [1]	2.4	0.7	0.108	0.062	0.30
		California					
1	[2] 200	Kelly Ball [3]	3.69	0.91	0.08	0.14	0.61
2	[2] 200	do	3.85	0.94	0.22	0.04	0.63
3	[2] 200	do	4.00	1.27	0.13	0.10	1.39
4	[4] 200	do	1.74	0.65	0.32	0.00	0.10

[1] Convereted to inches of slump. Conversion factor—Slump inches =0.59 Kelly Ball +1.02.

[2] Structural.

[3] Converted to inches of slump by calibration—1-i penetration of ball indicates 2 inches of slump.

[4] Pavement.

Table 9.—Analysis of variance, intermediate aggregate, percent passing ¾-inch si

Project No.	Range (*R*)	Mean ($\bar{X}$)	Material variance (σ_a^2)	Sampling variance (σ_s^2)	Testing variance (σ_t^2)	Standard deviation (σ)	Samples
	Pct	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Number*
1	79–98	92.60	4.25	0.00	8.12	3.52	200
2	33–89	69.09	122.92	5.59	4.54	11.54	200
3	34–92	71.52	124.04	9.31	24.46	12.56	200

Table 10.—Summary of statistical results on thickness of concrete pavement

	Samples	Mean ($\bar{X}$)	Overall variance (σ^2)	Standard deviation (σ)	Minimum	Maximu
8-inch uniform thickness						
Project number:	*Number*	*Inches*	*Inch*	*Inch*	*Inches*	*Inches*
1	34	8.66	0.192	0.435	7.63	9.53
2	39	8.42	0.171	0.415	7.61	9.13
3	48	8.35	0.040	0.200	7.86	8.80
4	58	8.36	0.077	0.276	7.76	9.49
5	61	8.05	0.035	0.185	7.66	8.59
6	66	8.11	0.089	0.300	7.46	8.78
7	73	8.06	0.112	0.333	7.58	9.58
Pooled values	----------	8.29	0.088	0.300	----------	----------
9-inch uniform thickness						
Project number:						
1	35	9.25	0.046	0.210	8.93	9.67
2	51	9.19	0.121	0.350	8.55	10.10
3	58	9.28	0.048	0.220	8.84	9.99
4	65	9.18	0.060	0.240	8.78	9.92
5	74	9.20	0.185	0.430	8.69	11.69
6	88	9.11	0.029	0.170	8.85	9.66
Pooled values	----------	9.20	0.083	0.290	----------	----------
10-inch uniform thickness						
Project number:						
1	64	10.38	0.061	0.240	9.41	10.91
2	124	10.34	0.079	0.280	9.82	11.48
3	132	10.35	0.079	0.230	9.75	10.94
4	141	10.28	0.083	0.290	9.63	11.27
Pooled values	----------	10.34	0.069	0.270	----------	----------

concentrated at the weaker points, and it is axiomatic that large variations in thickness are detrimental to the pavement. Uniformity of thickness will promote better slab action and, therefore, prolong pavement life.

Variation in concrete pavement thickness is shown by the data of figure 2, which is based on a report by the State of Michigan (*7*). The data depicted represent 656 cores taken from 15 projects from 1959–61. The historical data in table 10, extracted from a report by Louisiana (*8*), substantiates the variations shown in figure 2.

The variation shown in table 11 are from a statistical study of pavement thickness by the State of Oklahoma (*9*). The thicknesses were measured by a probe inserted in the plastic concrete. Little variation was exhibited in project 3; but as the mean was below the specified thickness, the pavement life expectancy was less than desired. Project 3 had a probable range of thickness from 7.8 to 10.2 inches, resulting in weak areas that would probably reduce the life of the pavement.

According to the high average concrete thicknesses reported in the Michigan and Louisiana studies, an excess of concrete is being placed by the contractors to avoid penalties. This same high variability–high average thickness relation has also been reported in other studies of thickness. Better control of placement not only could provide savings in concrete, but also produce pavement that is capable of better performance. Moreover, the development of a standard method for measuring the depth of plastic concrete, as placed, would aid in the control of thickness and eliminate expensive coring of the hardened pavement.

Variation in Portland Cement

The production of portland cements is being closely controlled by producers, according to the historical data on chemical analyses reported by several States. It is evident that State highway departments can reduce the testing of portland cement to at least the level recommended by ASTM in section 6 of ASTM–C–183–65T.

Conclusions

Variations in what is generally considered good construction has been shown by the research summarized here. However, the variations are of considerable magnitude and could be important factors in the performance of concrete structures. An awareness of these variations is insufficient; research must be undertaken to evaluate their effect and to develop procedures by which they can be reduced.

In many test results, much of the measured variation could be attributed to sampling and testing methods and procedures, and therefore the *real* variation may not be as large as results indicate. One of the major needs in concrete production is the development of better methods to measure the quality attributes of the concrete and the ingredients incorporated therein. Furthermore a clear delineation of responsibility should result in a more uniform product. It is the contractors' responsibility to produce quality material and the States' responsibility to measure the quality produced. Better measurement performance by the State highway departments will allow a more accurate estimate of product quality and provide a better basis for enforcement of the specifica-

Table 11.—Variation in pavement thickn probe method

Project No.	Observations	Standard deviation (σ)	Mean ($\bar{X}$)	Specif catio
	Number	*Inch*	*Inches*	*Inche*
1	72	0.3	8.5	8.0
2	95	0.1	8.9	9.0
3	100	0.4	9.0	9.0

tion requirements. This approach can result only in a product that is more uniform in character and has improved performance expectancy.

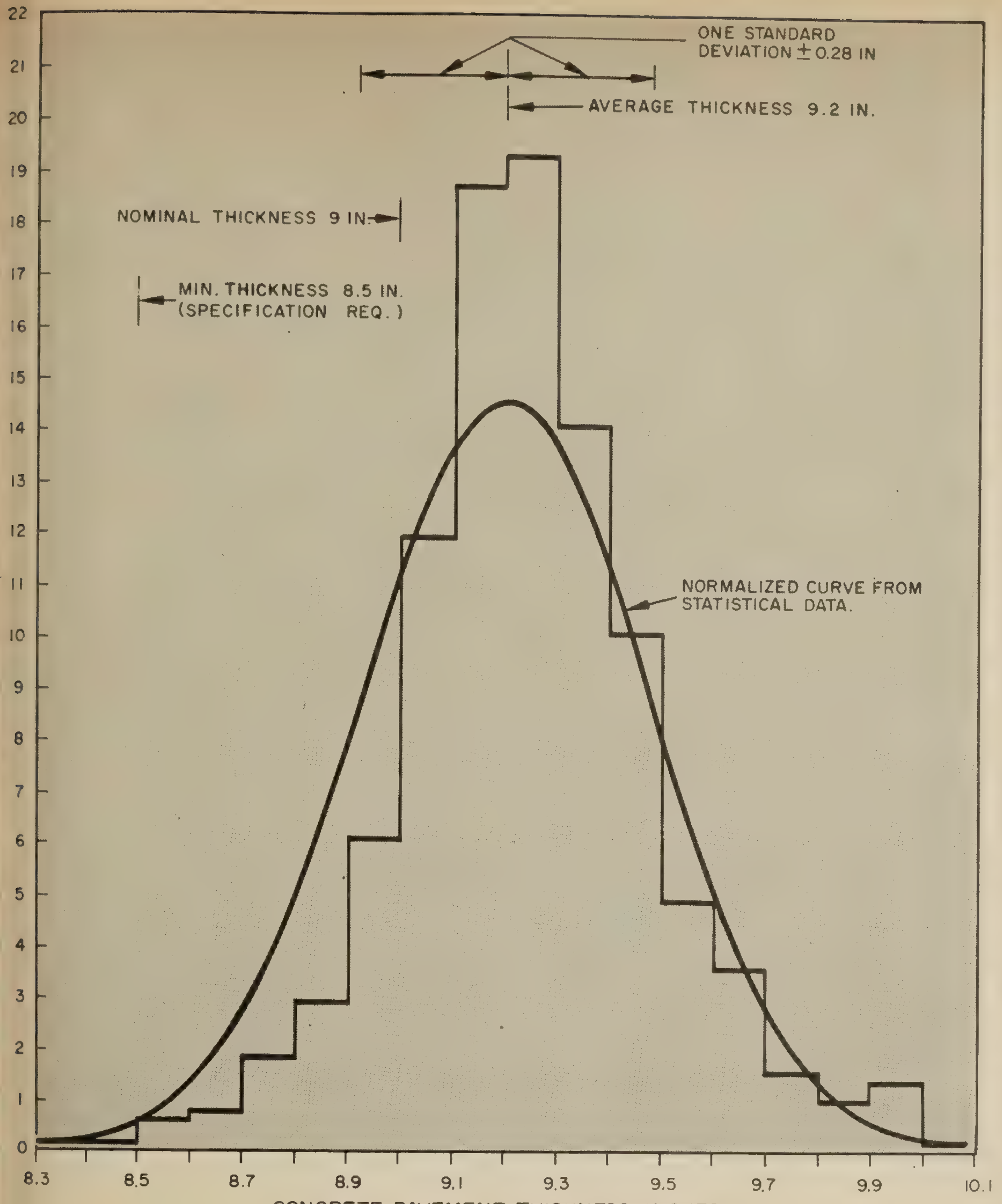

Figure 2.—Frequency distribution of concrete pavement thickness, 1959–61.

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Acquisition of Loading History Data on Highway Bridges

(*Continued from p. 183*)

an measured results from other recent
nilar investigations, specifically those con-
cted on eight bridges in Michigan and on
e bridge in Maryland.

- Because of the low-measured stress
nges at the Dumfries bridge, elements in
at particular bridge do not appear to be in
nger of fatigue failure caused by the
gular truck traffic.

The last conclusion is valid only if the
mpling can be termed representative of the
ıck traffic crossing the bridge; if it is
sumed that the occasionally allowed over-
ıds do not induce stresses in excess of the
ximum recorded ones, more than 3,500
s.i.; and if the slowly varying stresses
used by environmental changes do not
ntribute to fatigue distress.

ACKNOWLEDGMENTS

The Dumfries bridge study was conducted by members of the Bridges and Structures Group, Structures and Applied Mechanics Division, Bureau of Public Roads. The able assistance of Mr. Harry R. Laatz is especially acknowledged.

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Travel by Motor Vehicles in 1967

BY THE OFFICE OF PLANNING
BUREAU OF PUBLIC ROADS

Reported by W. JOHNSON PAG,
Highway Research Engine,
Current Planning Divisi

MOTOR vehicle travel in the United States in 1967 totaled 965.1 billion vehicle-miles, an increase of 3.7 percent over the travel in 1966. The travel data were compiled by the Bureau of Public Roads from information supplied by the State highway departments. Total travel for 1968, based on information for the first 9 months of the year, is estimated at 1 trillion, 10 billion vehicle-miles, a 4.7-percent increase over 1967.

The term vehicle-miles and the other technical terms used in this article are defined in the following statements:

Vehicle-miles.—Vehicle-miles refers to the amount of travel by one motor vehicle traveling 1 mile and includes travel on all highways and streets in the United States.

Trailer combinations.—A trailer combination is a truck or truck tractor pulling one or more trailers and/or a semitrailer.

Motor-fuel consumption.—Motor-fuel consumption is the total consumption of motor fuel by highway vehicles for the year, obtained from State records.

Motor-fuel consumption rate.—Motor-fuel consumption rate is the average rate of motor fuel usage in miles per gallon (m.p.g.).

The travel and related information for 1967 are shown in table 1 by road system and vehicle type. Total travel and travel by highway system are considered to be final figures, but because of incomplete data on which to make the distribution by vehicle type, the travel by vehicle type is subject to revision. Such data have been reported in PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH for a number of years; the latest for 1965 and 1966 appeared in vol. 34, No. 12, February 1968, pages 267–269.

Travel estimates by State and administrative highway system are shown in table 2. These are based on estimates prepared by the State highway departments and reported annually to the Bureau of Public Roads, beginning in 1966.

Table 1 has been adjusted to a new base for 1967 to bring it into agreement with table 2. Main rural roads travel in table 1 includes all travel on systems shown as 01, 31, 03, 05, and 09 in table 2. Travel on local rural roads includes travel in systems 07 and 11. Travel on urban streets consists of travel on all even numbered systems in table 2. Prior to 1967, parts of systems 05 and 09 had been classed as local rural roads.

Because of their intended principal use, the State estimates of 1967 travel were made according to a system classification and rural-urban distinction directly related to the Federal-aid program. In the Federal-aid law, an urban area is "an area including and adjacent to a municipality or other urban place having a population of 5,000 or more . . ." In the annual estimates reported in table 1 in the past, however, *urban* signified the areas within the political boundaries of municipa- ties such as cities, boroughs, and villages.

As State estimates are now being receiv annually, and because it is intended to u them as a base for table 1, it was decided redefine main rural roads, local rural roa and urban streets so that it would not necessary to split mileage and travel in a of the administrative highway systems.

These adjustments resulted in changes the proportionate distribution of mileage travel among the three highway categorie table 1. The main rural roads category chan from 35.2 percent of the total travel on percent of the mileage in 1966 to 37.3 perc of the travel on 15 percent of the mileage 1967. Local rural road travel was 14.3 perc of the total in 1966 on 72 percent of the t mileage, and 12.6 percent of the travel on percent of the mileage in 1967. In 1966, percent of the travel was on urban stre which comprised 14 percent of the t mileage. These proportions for 1967 are percent and 14 percent, respectively. Thus can be seen that the mileage shifted f urban streets to main rural roads had a m greater effect on the distribution of tr than on the distribution of mileage.

Passenger cars represented 81 percent o vehicles registered, and accounted for 80 cent of all travel in 1967; motorcycles, 2 cent of all vehicles registered and less th percent of all travel; and trucks and t combinations, 16 percent of all vehicles r

Table 1.—Estimated motor-vehicle travel in the United States and related data for calendar year 1967 [1]

	Motor-vehicle travel [2]					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Avera travel p gallon fuel consum
	Main rural roads	Local rural roads	All rural roads	Urban streets	Total			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Mile gallo*
Personal passenger vehicles:										
Passenger cars [3]	----------	----------	----------	----------	770, 971	80, 458	9, 582	55, 220	686	13.9
Motorcycles [3]	----------	----------	----------	----------	7, 737	1, 953	3, 962	103	53	75.0
All personal passenger vehicles	273, 332	94, 597	367, 929	410, 779	778, 708	82, 411	9, 449	55, 323	671	14.0
Buses:										
Commercial	1, 007	182	1, 189	1, 934	3, 123	90. 5	34, 508	667	7, 370	4.6
School	791	727	1, 518	338	1, 856	246. 7	7, 523	262	1, 062	7.0
All buses	1, 798	909	2, 707	2, 272	4, 979	337. 2	14, 766	929	2, 755	5.3
All passenger vehicles	275, 130	95, 506	370, 636	413, 051	783, 687	82, 748	9, 471	56, 252	680	13.9
Cargo vehicles:										
Single-unit trucks	63, 221	24, 426	87, 647	60, 021	147, 668	15, 364	9, 611	14, 491	943	10.1
Trailer combinations	21, 617	1, 400	23, 017	10, 760	33, 777	830	40, 695	6, 950	8, 373	4.8
All trucks	84, 838	25, 826	110, 664	70, 781	181, 445	16, 194	11, 204	21, 441	1, 324	8.4
All motor vehicles	359, 968	121, 332	481, 300	483, 832	965, 132	98, 942	9, 755	77, 693	785	12.4

[1] For the 50 States and District of Columbia.

[2] This table has been adjusted to a new base to bring it into agreement with table 2. Consequently, when the data is compared with that of 1966, a decrease in travel on local r shown for 1967, as are decreases in local rural road and urban street travel as percentages of total travel.

[3] Separate estimates of passenger car and motorcycle travel are not available by highway category.

Table 2.—Vehicle-miles, by State and highway system—1967

(Millions)

Division	State	Federal-aid highway system																		Not on Federal-aid systems				Sub-total rural	Sub-total urban and munici-pal	Total
		Interstate rural			Interstate urban			Sub-total Inter-state	Other primary			Secondary					Total Federal-aid rural	Total Federal-aid urban	Total Federal-aid	Other State rural 09	Other State munici-pal 10	Local rural 11	Local munici-pal 12			
		Final 01	Traveled 1/ 31	Total rural	Final 02	Traveled 1/ 32	Total urban		Rural 03	Urban 04	Total	State rural 05	State urban 06	Local rural 07	Local urban 08	Total										
New England	Connecticut	541	162	703	1,821	398	2,219	2,922	1,160	1,618	2,778	964	754	7	44	1,669	2,734	4,635	7,369	218	1,623	240	4,564	3,192	10,822	14,014
	Maine	495	49	544	63	67	130	674	1,367	430	1,797	817	124	-	-	941	2,728	684	3,412	856	291	278	387	3,862	1,362	5,224
	Massachusetts	1,047	174	1,221	1,288	540	1,828	3,049	2,418	4,501	6,919	611	523	1,059	1,306	3,499	5,309	8,158	13,467	145	873	705	6,579	6,159	15,610	21,769
	New Hampshire	393	59	452	61	37	98	550	1,094	295	1,389	730	125	3	2	860	2,279	520	2,799	135	170	109	437	2,523	1,127	3,650
	Rhode Island	59	44	103	489	103	592	695	237	1,139	1,376	206	390	16	206	818	562	2,327	2,889	79	157	125	911	766	3,395	4,161
	Vermont	210	189	399	30	63	93	492	752	141	893	305	1	143	19	468	1,599	254	1,853	8	*	195	209	1,802	463	2,265
	Total	2,745	677	3,422	3,752	1,208	4,960	8,382	7,028	8,124	15,152	3,533	1,917	1,228	1,577	8,255	15,211	16,578	31,789	1,441	3,114	1,652	13,087	18,304	32,779	51,083
Middle Atlantic	New Jersey	231	373	604	1,771	1,853	3,624	4,228	2,351	5,317	7,668	36	100	1,309	2,244	3,689	4,300	11,285	15,585	1,391	2,123	4,234	12,240	9,925	25,648	35,573
	New York	3,020	221	3,241	5,424	738	6,162	9,403	8,228	11,581	19,809	1,740	1,130	2,776	1,399	7,045	15,985	20,272	36,257	42	58	7,940	16,434	23,967	36,764	60,731
	Pennsylvania	3,613	1,171	4,784	2,205	894	3,099	7,883	7,789	5,908	13,697	5,516	3,326	46	93	8,981	18,135	12,426	30,561	3,164	4,010	3,996	10,392	25,295	26,828	52,123
	Total	6,864	1,765	8,629	9,400	3,485	12,885	21,514	18,368	22,806	41,174	7,292	4,556	4,131	3,736	19,715	38,420	43,983	82,403	4,597	6,191	16,170	39,066	59,187	89,240	148,427
South Atlantic (North)	Delaware	49	-	49	156	107	263	312	985	706	1,691	319	210	-	-	529	1,353	1,179	2,532	-	-	82	69	1,435	1,248	2,683
	Dist. of Col.	-	-	-	192	155	347	347	-	1,025	1,025	-	-	-	492	492	-	1,864	1,864	-	-	-	779	-	2,643	2,643
	Maryland	788	154	942	1,990	513	2,503	3,445	2,799	2,200	4,999	1,531	825	439	405	3,200	5,711	5,933	11,644	649	80	3,052	2,398	9,412	8,411	17,823
	Virginia	2,523	1,056	3,579	656	643	1,299	4,878	4,514	2,179	6,693	2,993	569	2,041	403	6,006	13,127	4,450	17,577	92	419	1,675	2,878	14,894	7,747	22,641
	West Virginia	308	523	831	78	192	270	1,101	2,030	719	2,749	1,491	116	870	32	2,509	5,222	1,137	6,359	7	24	301	1,160	5,530	2,321	7,851
	Total	3,668	1,733	5,401	3,072	1,610	4,682	10,083	10,328	6,829	17,157	6,334	1,720	3,350	1,332	12,736	25,413	14,563	39,976	748	523	5,110	7,284	31,271	22,370	53,641
South Atlantic (South)	Florida	1,598	1,092	2,690	1,251	799	2,050	4,740	4,990	3,777	8,767	4,293	2,577	206	33	7,109	12,179	8,437	20,616	325	1,463	6,217	4,145	18,721	14,045	32,766
	Georgia	1,630	1,322	2,952	1,332	353	1,685	4,637	5,440	1,798	7,238	2,506	514	1,251	420	4,691	12,149	4,417	16,566	139	359	1,766	5,391	14,054	10,167	24,221
	North Carolina	1,311	1,062	2,373	350	338	688	3,061	4,202	1,698	5,900	8,682	2,109	3	193	10,987	15,260	4,688	19,948	1,845	561	32	2,243	17,137	7,492	24,629
	South Carolina	985	716	1,701	85	159	244	1,945	4,200	1,383	5,583	3,083	440	117	5	3,645	9,101	2,072	11,173	307	926	345	335	9,753	3,333	13,086
	Total	5,524	4,192	9,716	3,018	1,649	4,667	14,383	18,832	8,656	27,488	18,564	5,640	1,577	651	26,432	48,689	19,614	68,303	2,616	3,309	8,360	12,114	59,665	35,037	94,702
East North Central	Illinois	2,365	1,678	4,043	4,094	772	4,866	8,909	8,365	6,928	15,293	1,078	604	2,405	541	4,628	15,891	12,939	28,830	1,472	3,604	2,792	13,160	20,155	29,703	49,858
	Indiana	2,092	1,226	3,318	969	990	1,959	5,277	6,007	2,627	8,634	2,831	623	1,875	711	6,040	14,031	5,920	19,951	120	193	1,133	6,002	15,284	12,115	27,399
	Michigan	2,575	245	2,820	2,540	1,414	3,954	6,774	6,584	5,667	12,251	1,351	417	6,513	992	9,273	17,268	11,030	28,298	37	54	4,415	12,250	21,720	23,334	45,054
	Ohio	3,682	541	4,223	3,353	1,364	4,717	8,940	7,965	5,483	13,448	4,268	1,460	2,320	1,959	10,007	18,776	13,619	32,395	101	427	4,725	12,074	23,602	26,120	49,722
	Wisconsin	1,346	132	1,478	541	9	550	2,028	5,400	2,230	7,630	1,685	570	1,510	215	4,680	10,073	4,265	14,338	41	62	1,155	5,335	11,269	9,662	20,931
	Total	12,060	3,822	15,882	11,497	4,549	16,045	31,928	34,321	22,935	57,256	11,213	3,674	14,623	5,118	34,628	76,039	47,773	123,812	1,771	4,340	14,220	48,821	92,030	100,934	192,964
West North Central	Iowa	1,346	211	1,557	203	84	287	1,844	4,944	1,362	6,306	-	-	1,522	313	1,835	8,023	1,962	9,985	80	51	886	2,471	8,989	4,484	13,473
	Kansas	810	216	1,026	310	78	388	1,414	3,760	1,045	4,805	574	23	1,417	410	2,424	6,777	1,866	8,643	86	64	1,080	2,442	7,943	4,372	12,315
	Minnesota	373	566	939	751	800	1,551	2,490	5,216	2,064	7,280	1,083	33	2,556	161	3,833	9,794	3,809	13,603	24	114	1,273	3,647	11,091	7,570	18,661
	Missouri	1,926	963	2,889	1,423	696	2,119	5,008	5,379	2,017	7,396	2,281	387	19	36	2,723	10,568	4,559	15,127	126	642	1,848	6,034	12,542	11,235	23,777
	Nebraska	675	185	860	59	46	105	965	2,862	628	3,490	636	39	635	71	1,381	4,993	843	5,836	4	-	1,002	1,865	5,999	2,708	8,707
	North Dakota	334	118	452	11	22	33	485	1,103	147	1,250	196	4	430	13	643	2,181	197	2,378	1	2	662	401	2,844	600	3,444
	South Dakota	398	265	663	18	9	27	690	1,563	195	1,758	258	31	378	26	693	2,862	279	3,141	22	7	576	452	3,460	738	4,198
	Total	5,862	2,524	8,386	2,775	1,735	4,510	12,896	24,827	7,459	32,285	5,029	517	6,957	1,030	13,532	45,198	13,515	58,713	343	880	7,327	17,312	52,868	31,707	84,575
East South Central	Alabama	837	860	1,697	119	721	840	2,537	3,793	1,616	5,409	1,335	259	1,361	194	3,149	8,186	2,909	11,095	41	50	753	3,083	8,980	6,042	15,022
	Kentucky	1,094	676	1,770	575	345	920	2,690	3,860	1,320	5,180	3,410	480	215	32	4,137	9,255	2,752	12,007	1,010	630	1,095	1,884	11,360	5,265	16,625
	Mississippi	517	472	989	119	569	688	1,677	3,595	552	4,247	803	82	970	182	2,037	6,357	1,504	7,861	18	13	635	1,293	7,011	2,810	9,821
	Tennessee	1,033	962	1,995	937	567	1,504	3,499	4,765	2,076	6,741	924	169	654	47	1,794	8,338	3,796	12,134	45	4	1,798	4,021	10,181	7,821	18,002
	Total	3,481	2,970	6,451	1,749	2,203	3,952	10,403	15,013	5,564	21,577	6,472	990	3,200	455	11,117	32,136	10,961	43,097	1,114	597	4,282	10,281	37,532	21,939	59,471
West South Central	Arkansas	629	383	1,012	287	38	325	1,337	2,595	923	3,519	2,075	318	250	37	2,690	5,943	1,603	7,546	58	68	666	1,374	6,667	3,045	9,712
	Louisiana	618	1,179	1,797	473	568	1,041	2,838	2,554	1,340	3,894	3,412	542	11	1	3,966	7,774	2,924	10,698	616	375	661	2,289	9,051	5,588	14,639
	Oklahoma	1,169	266	1,435	682	222	904	2,339	4,581	1,422	6,003	978	282	712	247	2,219	7,706	2,855	10,561	208	113	975	3,282	8,889	6,250	15,139
	Texas	3,018	1,606	4,624	6,036	1,965	8,002	12,625	11,311	6,927	18,238	7,626	2,079	-	-	9,705	23,561	17,008	40,569	1,250	920	2,473	13,790	27,284	31,718	59,002
	Total	5,434	3,434	8,868	7,479	2,794	10,272	19,140	21,042	10,612	31,654	14,091	3,221	983	285	18,580	44,984	24,390	69,374	2,132	1,476	4,775	20,735	51,891	45,601	98,492
Mountain	Arizona	853	1,354	2,207	331	207	538	2,745	1,174	285	1,459	499	48	473	1,899	2,919	4,353	2,770	7,123	195	15	373	1,519	4,911	4,304	9,215
	Colorado	1,213	167	1,380	646	90	736	2,116	2,033	1,485	3,518	917	144	-	-	1,061	4,330	2,365	6,695	19	48	1,272	2,509	5,621	5,022	10,643
	Idaho	397	304	701	27	68	95	796	1,191	203	1,394	325	24	303	52	704	2,520	374	2,894	3	2	671	525	3,194	902	4,096
	Montana	134	667	801	26	25	51	852	1,357	224	1,581	400	53	-	-	453	2,558	328	2,886	-		734	460	3,292	788	4,080
	Nevada	480	237	717	20	87	107	824	630	237	867	265	53	96	165	579	1,708	562	2,270	22	20	100	650	1,830	1,232	3,062
	New Mexico	792	570	1,362	140	208	348	1,710	1,375	519	1,894	803	129	5	*	938	3,545	996	4,542	72	437	395	674	4,014	2,107	5,121
	Utah	146	982	1,128	143	385	528	1,656	790	419	1,209	537	438	98	132	1,205	2,553	1,517	4,070	35	125	403	625	2,992	2,268	5,260
	Wyoming	446	321	767	13	21	34	801	813	125	938	210	7	13	9	239	1,803	175	1,978	2	3	275	324	2,080	502	2,582
	Total	4,471	4,602	9,073	1,351	1,076	2,427	11,500	9,363	3,497	12,860	3,956	906	979	2,257	8,098	23,371	9,087	32,458	339	651	4,224	7,387	27,934	17,125	45,059
Pacific	California	2,986	4,309	7,295	10,112	4,028	14,140	21,435	11,374	14,412	25,786	2,457	844	3,547	3,799	10,647	24,673	33,195	57,868	594	1,197	7,680	36,479	32,947	70,871	103,818
	Oregon	1,344	112	1,456	693	30	723	2,179	2,251	1,333	3,584	1,034	283	723	456	2,496	5,464	2,795	8,259	36	14	1,370	1,896	6,870	4,695	11,565
	Washington	597	857	1,454	1,317	561	1,878	3,332	2,227	1,305	3,532	999	521	1,527	942	3,989	6,207	4,646	10,853	15	144	2,826	3,902	9,049	8,692	17,741
	Total	4,927	5,278	10,205	12,122	4,619	16,741	26,946	15,852	17,050	32,902	4,490	1,648	5,797	5,197	17,132	36,344	40,636	76,980	645	1,355	11,875	42,277	48,866	84,258	133,124
Total - All Divisions		55,036	30,997	86,033	56,214	24,928	81,142	167,175	175,074	113,531	288,605	80,973	24,784	42,825	21,638	170,225	385,805	241,100	626,905	15,747	22,536	77,996	218,354	479,548	481,990	961,538
	Alaska	-	-	-	-	-	-		305	101	406	105	35	-		141	411	136	547			107	219	518	355	873
	Hawaii	16	108	124	102	258	360	484	473	402	875	233	53			286	830	815	1,645	-	-	404	672	1,234	1,487	2,721
United States Total		55,052	31,105	86,157	56,316	25,186	81,502	167,659	175,852	114,034	289,886	81,311	24,872	42,825	21,638	170,652	387,046	242,051	629,097	15,747	22,536	78,507	219,245	481,300	483,832	965,132

1/ Traveled-way includes travel on highways in the Interstate System status of completion group 2. This group consists of mileage which is adequate for present traffic but will need further construction and improvement to bring it to full Interstate standards. In some instances these are divided highways with full control of access and in others they are two, three or four-lane undivided highways with no access control.

* Less than 0.5 million vehicle-miles.

Note: For this table travel on frontage roads has been placed in systems 11 and 12.

tered and 19 percent of all travel. Similar figures for buses were less than 1 percent.

Average performance for all vehicles in 1967 differed somewhat from that reported in 1966. The average motor vehicle traveled 9,755 miles in 1967, half of it in cities, and consumed 785 gallons of fuel at a rate of 12.42 miles per gallon. The average passenger car traveled 9,582 miles and consumed 686 gallons of fuel at a rate of 13.96 miles per gallon.

According to the State estimates, the traveled way of the Interstate System carried 167.7 billion vehicle-miles, or 17.4 percent of the total 1967 travel on all roads and streets. The traveled way consisted of 22,288 miles of Interstate System highways now in use and of 18,712 miles of existing connecting highways. Service for this total mileage will be provided by the Interstate System when it is completed. From the State estimates it is expected that by 1975 the 41,000 mile Interstate System, comprising little more than 1 percent of the total road and street mileage of the United States, will carry more than 20 percent of the total 1,213 billion miles of travel estimated for 1975.

According to the State estimates of 197 travel, all Federal-aid systems combined, which includes about 25 percent of all roads and streets, carried 65 percent of all travel.

NEW PUBLICATIONS

The Bureau of Public Roads has recently published two documents. These publications may be purchased from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402, prepaid. The following paragraphs give a brief description of each publication and its purchase price.

Fatal and Injury Accident Rates on Federal-Aid and Other Highway Systems, 1967

Fatal and Injury Accident Rates on Federal-Aid and Other Highway Systems, 1967 (45 cents a copy), is a 36-page publication and is the first in a contemplated annual series to present accident, fatality and injury rates per 100 million vehicle-miles by State and administrative highway system. Included also are similar rates based on numbers of registered vehicles, licensed drivers, and population. In addition to the rates, the actual numbers of highway miles in service, vehicle miles, accidents, fatalities, and injuries are shown in supporting tables. This compilation is based on reports submitted by the State highway departments.

Standard Plans for Highway Bridges, Volume IV, Typical Continuous Bridges, 1969

Standard Plans for Highway Bridges, Volume IV, Typical Continuous Bridges, 1969 ($1.50 a copy), is a revision of the 1962 edition with respect to bridge widths and current design specifications. These plans are intended to serve as a useful guide in the development of suitable and economical bridge designs. An effort has been made to give sufficient information on all plans so that they may be readily modified in the preparation of contract drawings.

The volume contains four sets of plans for substructure and superstructure of four-span continuous bridges, including a concrete voided slab, a concrete T–beam, a concrete box girder, and a composite welded steel girder.

A variety of bent and footing types have been detailed for the four bridges. Intermediate supports include monolithic bents, framed bents, and solid wall piers. Footing types include pedestal piles, steel bearing piles, friction piles, and spread footings.

Bridge roadway width is 44 feet with S 20–44 live loading for the standard 2-lane, two-directional roadway.

HIGHWAY RESEARCH AND DEVELOPMENT REPORTS AVAILABLE FROM CLEARINGHOUSE FOR FEDERAL SCIENTIFIC AND TECHNICAL INFORMATION

As the results of research are useful only when they reach those who can implement them or apply the knowledge gained to other endeavors, the problem of how to make research reports available to interested persons has always been a concern of administrators. This problem was answered, at least in part, by the establishment of the Clearinghouse for Federal Scientific and Technical Information, a repository for technical documents organized in 1964 on the foundations of the Department of Commerce's Office of Technical Services.

The Clearinghouse collects research reports that are the results of work performed by Government laboratories or by industrial firms and private institutions under contract to sponsoring Federal agencies. More than 50 departments and agencies supply 50,000 reports to the Clearinghouse each year. The Clearinghouse announces, reproduces, and sells the reports at a nominal cost. Many organizations use them to produce new items for management, improve production processes, reduce costs, solve technical problems, prepare bids on Government contracts, or keep abreast of the state-of-the-art.

Among the documents collected by the Clearinghouse are the published results of federally supported highway research and development projects. As with all the documents on file, these publications are available as either microfiche or paper copies. Microfiche is a 4- by 6-inch sheet of film that contains as many as 70 pages of a document. Paper copy is produced by offset printing. Microfiche copies are more economical than paper copies, are easier to handle, and can be filed readily.

As a service to the readers of PUBLIC ROADS, A JOURNAL OF HIGHWAY RESEARCH, listings of these highway research and development publications available from the Clearinghouse will be published, as space permits in future issues. These publications will be listed chronologically beginning with the first report collected by the Clearinghouse and will continue until all the highway research and development publications on file at the Clearinghouse have been listed. From that point, each issue will list the reports that will have been collected by the Clearinghouse since the last issue. Each listing will include the accession (stock number) of each report. Any highway research and development publication can be purchased by sending the stock number and a check or money order to Clearinghouse for Federal Scientific and Technical Information, Sills Building, 5 Port Royal Road, Springfield, Va. 22 1. Paper copies are priced at $3 each and microfiche copies at 60 cents each. The Clearinghouse requires prepayment on all orders.

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PUBLICATIONS of the Bureau of Public Roads

A list of the more important articles in PUBLIC ROADS *and title eets for volumes 24–34 are available upon request addressed to reau of Public Roads, Federal Highway Administration, U.S. partment of Transportation, Washington, D.C. 20591.*

The following publications are sold by the Superintendent of cuments, Government Printing Office, Washington, D.C. 20402. ders should be sent direct to the Superintendent of Documents. epayment is required.

cidents on Main Rural Highways—Related to Speed, Driver, and Vehicle (1964). 35 cents.

gregate Gradation for Highways: Simplification, Standardization, and Uniform Application, and A New Graphical Evaluation Chart (1962). 25 cents.

nerica's Lifelines—Federal Aid for Highways (1966). 20 cents.

pacity Analysis Techniques for Design of Signalized Intersections (Reprint of August and October 1967 issues of PUBLIC ROADS, a Journal of Highway Research). 45 cents.

nstruction Safety Requirements, Federal Highway Projects (1967). 50 cents.

rrugated Metal Pipe Culverts (1966). 25 cents.

eating, Organizing, & Reporting Highway Needs Studies (Highway Planning Technical Report No. 1) (1963). 15 cents.

ital and Injury Accident Rates on Federal-Aid and Other Highway Systems, 1967. 45 cents.

deral-Aid Highway Map (42 x 65 inches) (1965). $1.50.

deral Laws, Regulations, and Other Material Relating to Highways (1965). $1.50.

deral Role in Highway Safety, House Document No. 93, 86th Cong., 1st sess. (1959). 60 cents.

eeways to Urban Development, A new concept for joint development (1966). 15 cents.

idelines for Trip Generation Analysis (1967). 65 cents.

andbook on Highway Safety Design and Operating Practices (1968). 40 cents.

ighway Beautification Program. Senate Document No. 6, 90th Cong., 1st sess. (1967). 25 cents.

ighway Condemnation Law and Litigation in the United States (1968):
Vol. 1—A Survey and Critique. 70 cents.
Vol. 2—State by State Statistical Summary of Reported Highway Condemnation Cases from 1946 through 1961. $1.75.

ighway Cost Allocation Study: Supplementary Report, House Document No. 124, 89th Cong., 1st sess. (1965). $1.00.

ighway Finance 1921–62 (a statistical review by the Office of Planning, Highway Statistics Division) (1964). 15 cents.

ighway Planning Map Manual (1963). $1.00.

ighway Research and Development Studies. Using Federal-Aid Research and Planning Funds (1967). $1.00.

ighway Statistics (published annually since 1945):
1965, $1.00; 1966, $1.25; 1967, $1.75.
(Other years out of print.)

ighway Statistics, Summary to 1965 (1967). $1.25.

ighway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

ighways and Human Values (Annual Report for Bureau of Public Roads) (1966). 75 cents.
Supplement (1966). 25 cents.

ighways to Beauty (1966). 20 cents.

ighways and Economic and Social Changes (1964). $1.25.

ydraulic Engineering Circulars:
No. 5—Hydraulic Charts for the Selection of Highway Culverts (1965). 45 cents.
No. 10—Capacity Charts for the Hydraulic Design of Highway Culverts (1965). 65 cents.
No. 11—Use of Riprap for Bank Protection (1967). 40 cents.

Hydraulic Design Series:
No. 2—Peak Rates of Runoff From Small Watersheds (1961). 30 cents.
No. 3—Design Charts for Open-Channel Flow (1961). 70 cents.
No. 4—Design of Roadside Drainage Channels (1965). 65 cents.

Identification of Rock Types (revised edition, 1960). 20 cents.
Request from Bureau of Public Roads. Appendix, 70 cents.

The 1965 Interstate System Cost Estimate, House Document No. 42, 89th Cong., 1st sess. (1965). 20 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.
Amendment No. 1 to above (1966). $1.00.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
Part V only of above—Traffic Controls for Highway Construction and Maintenance Operations (1961). 25 cents.

Maximum Desirable Dimensions and Weights of Vehicles Operated on the Federal-Aid Systems, House Document No. 354, 88th Cong. 2d sess. (1964). 45 cents.

Modal Split—Documentation of Nine Methods for Estimating Transit Usage (1966). 70 cents.

National Driver Register. A State Driver Records Exchange Service (1967). 25 cents.

Overtaking and Passing on Two-Lane Rural Highways—a Literature Review (1967). 20 cents.

Presplitting, A Controlled Blasting Technique for Rock Cuts (1966). 30 cents.

Proposed Program for Scenic Roads & Parkways (prepared for the President's Council on Recreation and Natural Beauty), 1966. $2.75.

Reinforced Concrete Bridge Members—Ultimate Design (1966). 35 cents.

Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles (1964). 45 cents.

Role of Economic Studies in Urban Transportation Planning (1965). 45 cents.

The Role of Third Structure Taxes in the Highway User Tax Family (1968). $2.25.

Standard Alphabets for Highway Signs (1966). 30 cents.

Standard Land Use Coding Manual (1965). 50 cents.

Standard Plans for Highway Bridges:
Vol. I—Concrete Superstructures (1968). $1.25.
Vol. II—Structural Steel Superstructures (1968). $1.00.
Vol. IV—Typical Continuous Bridges (1969). $1.50.
Vol. V—Typical Pedestrian Bridges (1962). $1.75.

Standard Traffic Control Signs Chart (as defined in the Manual on Uniform Traffic Control Devices for Streets and Highways) 22 x 34, 20 cents—100 for $15.00. 11 x 17, 10 cents—100 for $5.00.

Study of Airspace Utilization (1968). 75 cents.

Traffic Safety Services, Directory of National Organizations (1963). 15 cents.

Typical Plans for Retaining Walls (1967). 45 cents.

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